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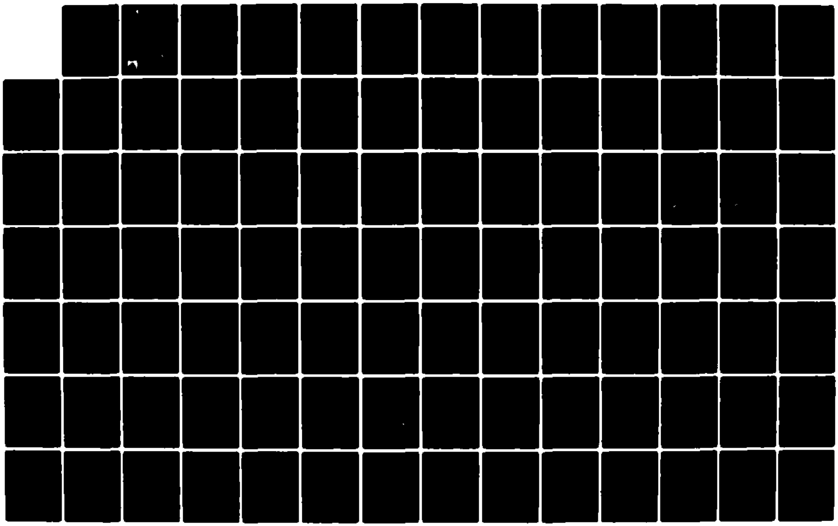

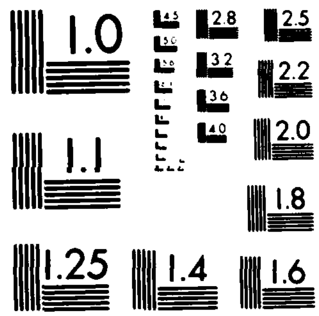

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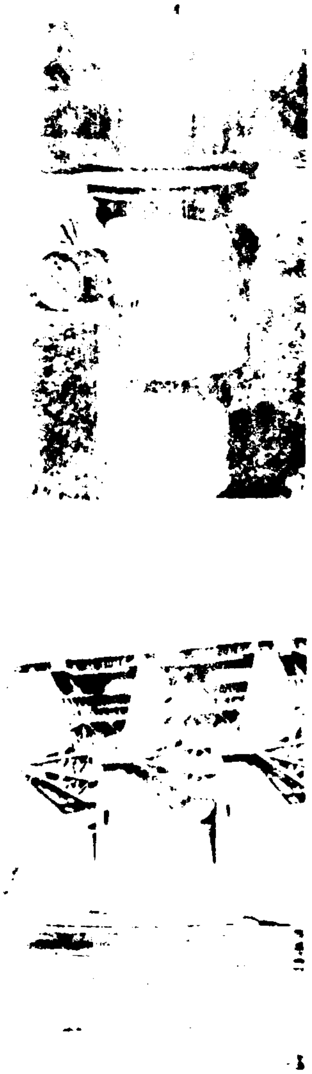

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PROCEEDINGS OF SYMPOSIUM ON COST ESTIMATING FOR WATER SUPPLY PLANNING STUDIES

Thomas M. Walski, Editor

Environmental Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

September 1983

Final Report

Approved For Public Release; Distribution Unlimited

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Prepared for Office, Chief of Engineers, U. S. Army
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Under Water Conservation and Supply Program
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ENVIRONMENTAL LABORATORY

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number) This report contains the papers presented at the symposium "Cost Estimating for Water Supply Planning Studies" as conducted at the ASCE Specialty Conference, Water Supply--The Management Challenge, in Tampa, Florida, 14-16 March 1983. The proceedings include an overview chapter, description of alternative approaches from the detailed "systems estimate" to the more general approach of "parametric estimating." Other topics include verification of estimating (Continued)		

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20. ABSTRACT (Continued).

models, spatial cost estimating, estimating for small systems and large regional studies, and a method for estimating surface water intake costs.

PREFACE

These proceedings are being published under the Water Conservation and Supply Program, Information Transfer and Analysis work unit. The papers were collected for publication by the symposium chairman, Dr. Thomas M. Walski, of the Water Resources Engineering Group (WREG), Environmental Engineering Division (EED), Environmental Laboratory (EL), U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss.

The publication was assembled under the supervision of Mr. Michael R. Palermo, Chief, WREG, Mr. Andrew J. Green, Chief, EED, and Dr. John Harrison, Chief, EL.

Commander and Director of WES was COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

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OVERVIEW

Cost estimating for water resources planning studies is a neglected area in water resources planning and management. Realizing the need to increase awareness in the problems involved with planning level estimates and the analytical tools available to assist in making these estimates, the Water Systems Committee of the Water Resources Planning and Management Division of the American Society of Civil Engineers sponsored a mini-symposium on Cost Estimating for Water Supply Planning Studies. This symposium was held at the specialty conference, Water Supply - The Management Challenge, in Tampa, Florida, 14-16 March 1983. The papers presented at that symposium are included in this proceedings to further increase this awareness among practicing engineers, planners, and estimators.

To set the stage for the remaining papers, Thomas Walski from the U. S. Army Engineer Waterways Experiment Station (WES) presented the paper entitled, "Planning Level Cost Estimating--Science, Art, or Witchcraft?" which summarizes the state of the art and describes some important issues in planning level estimates in water resources studies.

The more traditional cost estimating approach involves quantity take-offs from detailed designs with unit prices determined using standard estimating methods such as those provided by the R. S. Means Company. Dwayne Lehigh of Means presents techniques for developing a "systems estimate" which lie between detailed estimates and what Lehigh calls "order-of-magnitude" estimates.

An important topic of recent importance is determination of water supply cost as a function of the location in a system. This is known as "spatial" costing and is discussed by Donald Schlenger of the Hackensack (N. J.) Water Utility.

Before confidence can be placed in any model, it is necessary to verify the model by comparing the predicted results with the actual system to be modeled. Models of cost are no exception and Thomas Walski and Anita Lindsey of the WES describe verification of the Corps' MAPS (Methodology for Areawide Planning Studies) computer program which calculates costs as one of its functions.

Small water systems present special problems to cost estimators. Arun Deb and William Richards of Roy F. Weston, West Chester, Pa., describe their approach to estimating for small water systems.

Developing a generalized method for estimating cost is not a simple procedure, but an effort that must be approached carefully and systematically. Ken Cable of CH2M-Hill and Janet Condra of WES present a method for calculating costs for surface-water intake structures.

The cost estimating models developed for water resources studies are a subset of more general estimating models. Keith Burbridge of BFH Parametrics, Mountain View, Calif., describes an approach called "parametric analysis" which can generate cost estimates for a wide array of facilities and products.

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THIS PAPER IS A COMPONENT PART OF THE FOLLOWING COMPILATION REPORT:

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P001 907	Developing Cost Estimating Methods for Surface-Water Intake Structures.
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Presenting the estimates developed for a large number of alternatives in a regional study entails some special problems. Marshall Lee of the Benham Group, Oklahoma City, Okla., was unable to attend the conference, but submitted this paper describing how cost estimates were presented in the Tulsa Urban Study.

Richard Eilers and Robert Clark of the Environmental Protection Agency (EPA) described the work being done by their agency to better understand the nature of water supply costs. Clark and Richard Miles of RMM Services discussed their experiences with spatial cost estimating. Because of the time delay involved in obtaining clearance for the EPA papers, they could not be included in these proceedings.

A paper by James Heaney, Khlifa Maalel, and Carol Merkel, University of Florida, Gainesville, was presented at the conference but is not included in the proceedings.

The symposium ended with a panel discussion, "How Not To Prepare Planning Level Estimates." Copies of a transcript of the discussion are available from the editor.

CONTENTS

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PLANNING LEVEL COST ESTIMATING--SCIENCE, ART, OR WITCHCRAFT?

Thomas M. Walski, A.M. ASCE

Background

In reviewing a water supply master plan, a reader normally finds detailed data on water source yields and future system layout, but very little on costs of implementing the plan. One often gets the impression that these cost estimates, which usually include only capital costs, were inserted into the master plan as an afterthought. There is frequently little or no documentation indicating that life-cycle costing or a cost-benefit analysis was used to select the master plan over alternative plans. As a result, cost estimating frequently seems to be regarded as an appendage rather than an integral part of the planning process.

Unlike detailed cost estimating performed for construction contracts, on which the future of entire firms depend, estimating in planning studies often appears to be approached in a fairly haphazard manner. The recipe is simple--update an old pump station estimate, pull some numbers out of a reference book for pipe costs, find a cost function for treatment plants in an EPA report, mix with a heavy dose of "judgement" and voila, the estimate is complete. After all, it's just a planning estimate.

While most engineering schools offer courses or portions of courses on construction estimating, students are left to learn planning estimating on their own--often by talking with the person who performed the firm's last planning level estimate. Very few estimating books mention planning level estimates and in those which do, a caveat is usually included which reads something like "Considerable experience and judgement are required to obtain a dependable approximate estimate..." (Peurifoy, 1975). While this statement is true, estimates for planning studies generally seem to rely too much on judgement. "Judgement" is often used as a substitute for consistency and rigor.

Haveman (1972) reported on ex post studies of planning level cost estimates for large Corps of Engineers and Bureau of Reclamation flood control, navigation and hydropower projects. He found the cost estimates to be of widely varying accuracy. Although Haveman's interests were more in evaluating benefit-cost analyses to determine if (ex ante) predicted benefits and costs were realized (ex post), he found "enormous inconsistency in achieving accurate cost estimates." This type of analysis has not been performed for smaller scale water supply projects.

In general very little work was done prior to the 1970's on evaluating costs at the planning level. Some notable exceptions are the U. S. Bureau of Reclamation (1959), Illinois State Water Survey Reports (Dawes, 1970; Gibb and Sanderson 1969), Stoltenberg (1969), Koenig (1966, 1967), Linaweaver and Clark (1964), and Orlob and Lindorf (1958). With the increase in environmental awareness, the Environmental Protection Agency invested significant amounts of money in collecting cost data, primarily for wastewater treatment and collection

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(Patterson and Banker, 1971, Smith and Eilers, 1973; Van Note, et al, 1975; Pound, Crites and Criffes, 1975; Benjes, 1979; and Office of Water Program Operations, 1978, 1980A, 1980B).

With the Safe Drinking Water Act, EPA investigated the costs of water supply (Clark, et al, 1978; Gumerman, Culp and Hansen, 1979; U. S. EPA, 1978; Clark, 1979; Clark and Stevie, 1981A, 1981B; Clark and Morand, 1981; Clark, Stafford, and Goodrich, 1982; Clark and Dorsey, 1982). The U. S. Army Corps of Engineers, initially through the Urban Studies Program, also realized the usefulness of a centralized codified cost estimating procedure. The Corps, therefore, developed the CAPDET (Computer Assisted Procedure for Design and Evaluation of Wastewater Treatment Facilities) Computer Program (U. S. Army, 1978; Harris, Cullinane and Sun, 1978) for wastewater treatment and the MAPS (Methodology for Areawide Planning Studies) Computer Program (U. S. Army, 1980; Walski;, 1980A, 1981; Walski and Pelliccia, 1981; Lindsey and Walski, 1982) for many other water resources facilities. Dickson, (1978) published tables of costs for actual water supply facilities constructed since 1949. Hinomoto (1977) and Whitlatch and Asplund (1981) also published cost functions for water supply facilities, and rural water distribution systems respectively.

Advances in planning level cost estimating have generally resulted from needs of specific studies. Only since the 1960's have generalized cost estimating procedures, to be utilized by a large number of users for a wide range of problems, been developed.

An important property of estimates developed using these procedures is that they are "reproducible." That is, different individuals making the estimates independently, using the same initial data, will produce the same results. This is not always the case for estimating methods relying heavily on "judgement." With such procedures, different estimators (or even the same estimator on different days) can arrive at different results.

Another interesting phenomena is that development of planning type estimating procedures is not being directed by estimators, but rather by planners and engineers who need to use the estimates. In general, estimators appear to have an aversion to publishing their "secrets", and are very uncomfortable with making estimates without having detailed plans and specifications for the facility under consideration--a luxury which is not available during planning studies when only the crudest information about a facility is likely to be known.

The trend of developing and applying rigorous, life-cycle cost estimating techniques is continuing and a science of cost estimating for planning studies is evolving. This conference represents an important step in that evolution.

Overview

In this paper, planning level cost estimating is defined, its characteristics are described, and the phases of preparing a planning level estimate and the interactions between the phases are discussed. The problem of tradeoffs between detail and accuracy is analyzed, cost functions and the data used in preparing them are presented and, finally, the relationship between planning and estimating is discussed.

Definition

A planning level cost estimate is a prediction of the capital and operation and maintenance cost of a facility based on information which is less detailed than the plans and specifications. Planning level estimates are normally used for budgetary and screening purposes. As such, the estimate should be easy to develop and require very little additional information beyond what a planner normally has available. Therefore, good planning level cost estimating procedures should: 1.) require input data that are easily obtainable, 2.) be reproducible, 3.) be easy to use, 4.) be capable of considering a wide variety of alternatives, and 5.) be sufficiently accurate for the intended application.

Phases

There are several phases in preparing and using a cost estimate as shown in Figure 1. First, historic data are gathered and converted into cost functions. The functions are then applied using data describing the current project under consideration. (These steps are discussed more in later sections.) The estimate thus generated is used in the design, screening, or plan evaluation process.

Estimates based on detailed plans and specifications involve one pass through the flowchart in Figure 1. In planning studies, the project data are not in final form, but instead, are modified based on the results of previous estimates and the need to screen a wide variety of different alternatives. It is this iterative nature of planning estimating (as shown in Figure 2) that distinguishes it from estimates based on detailed designs. Not only are the project features not known with any detail at the planning stage, they are not known with any certainty.

The cost functions used must be accurate over a wide range of values for the project data. For example, if an estimate for a pump is being developed based on detailed plans and specifications, the pump characteristics are known and it is possible to use "cost = $40,000" as a cost function. In a planning study, the cost function must be considerably more general to account for the fact that the discharge and head produced by the pump may vary for each alternate.

In some applications, cost estimating procedures are often buried deep inside of optimization models (e.g. linear programmming) as shown in Figure 3. When the estimating procedure is obscured from the planner, great care must be taken to insure that the estimating procedure is realistic. For example, a linear programming model is only reliable over the domain of the decision variables for which the cost are linear. Unfortunately, operations research analysts can become so involved with their optimization models that they overlook the need for accurate cost functions and end up playing a game called GIGO (Garbage In--Garbage Out).

Accuracy

Tradeoff. **The more time, effort and design data put into preparing an estimate, the more accurate the estimate will be. However, there are diminishing returns in this process since accuracy is limited by the quality of the**

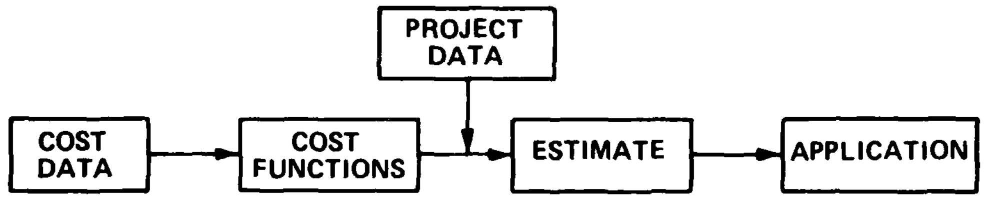

Fig. 1. Typical Cost Estimating Procedure

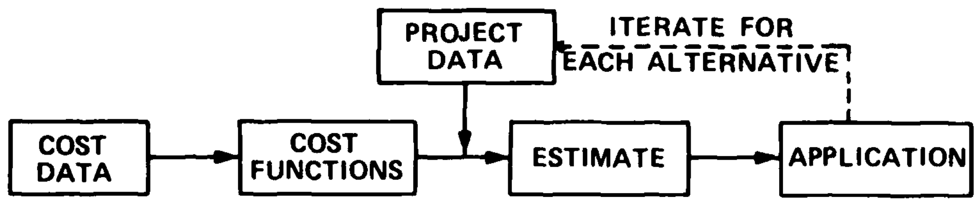

Fig. 2. Cost Estimating in Planning Study

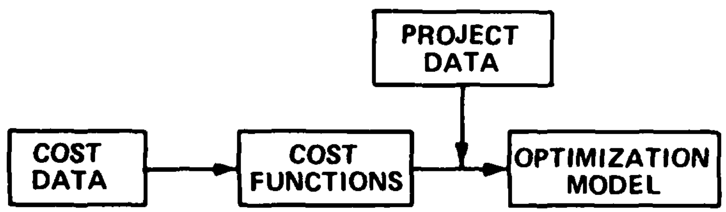

Fig. 3. Cost Estimating in Optimization Model

design data. The relationship between accuracy and time spent on the estimate for different levels of input data is represented in Figure 4.

While the accuracy increases less than proportionally with the time spent on the estimate, the cost increases proportionally. This is shown as the straight line in Figure 5. Thus, it is evident that there is a tradeoff between accuracy and ease of preparing an estimate. It is worth increasing the effort (cost) spent on making an estimate as long as the value of the increased accuracy exceeds the increased cost. This accuracy-minus-effort function is shown as the dashed line in Figure 5. The optimal time corresponds to the maximum point on the dashed curve. It should also be noted that as the detail of the design data increases, it becomes worthwhile to spend more time working on the estimate.

Of course it is not really possible to quantify the value of accuracy in terms of dollar values as shown in Figures 4 and 5. Nevertheless, Figures 4 and 5 should serve as a conceptual model illustrating: 1.) the law of diminishing returns as applied to preparing estimates, 2.) estimates are limited by the input data, 3.) there is an optimal amount of time to be spent on preparing an estimate, and 4.) it is worth spending more time refining estimates based on detailed data than on rough data.

Selecting Detail. It is not possible to determine the correct level of detail to be used in an estimating procedure, using the analysis described above. How then, does one determine the correct level of detail? The method this author has found to be useful consists of developing a "wiring diagram" such as that shown for a surface water intake system in Figure 6. For this example, the final answer desired is the average annual cost. It is not possible to simply determine the average annual cost of a surface water intake as a function of say, design flow, with any accuracy. Instead one must divide the average annual cost into separate cost items. In the first step these separate items are capital and O & M costs. These costs can be divided into smaller and smaller items until an item is reached for which a simple yet accurate cost function can be prepared.

In this example, a single cost function could not be developed for the construction of intake structures. Rather, separate functions were needed for exposed towers and submerged intakes, to which additional costs could be added for cofferdams and pilings required for difficult subsoils. For each item in the diagram the individual developing the procedure must decide if it is possible to write a cost function for that item or divide it into several items. This can best be done by making a list of the variables on which the cost of the item depends. If this list is fairly long, then it is usually not possible to derive at cost function to explain the variation in costs. As the process proceeds, one expands the diagram to the right until there are no items which cannot be explained by cost functions based on data available to the planner. Conversely, the diagrams can also be used to identify the kind of information a planner must gather to use the procedure. These parameters are in the boxes on the right end of the diagram.

Problems occur when, in order to make a good estimate, one must know the value of a design or operating parameter, which is not usually known in a planning study. For the example in Figure 6, the cost of a bridge to the intake

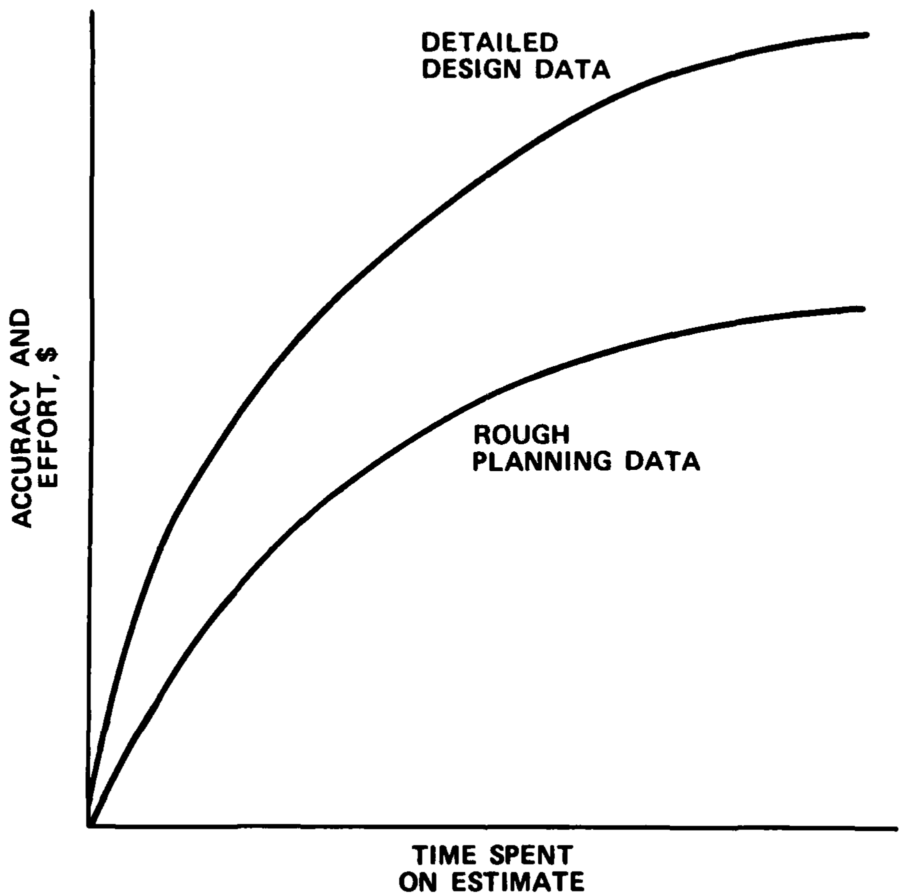

Fig. 4. Relationship between Time Spent on Estimate and Value of Accuracy for Two Types of Data

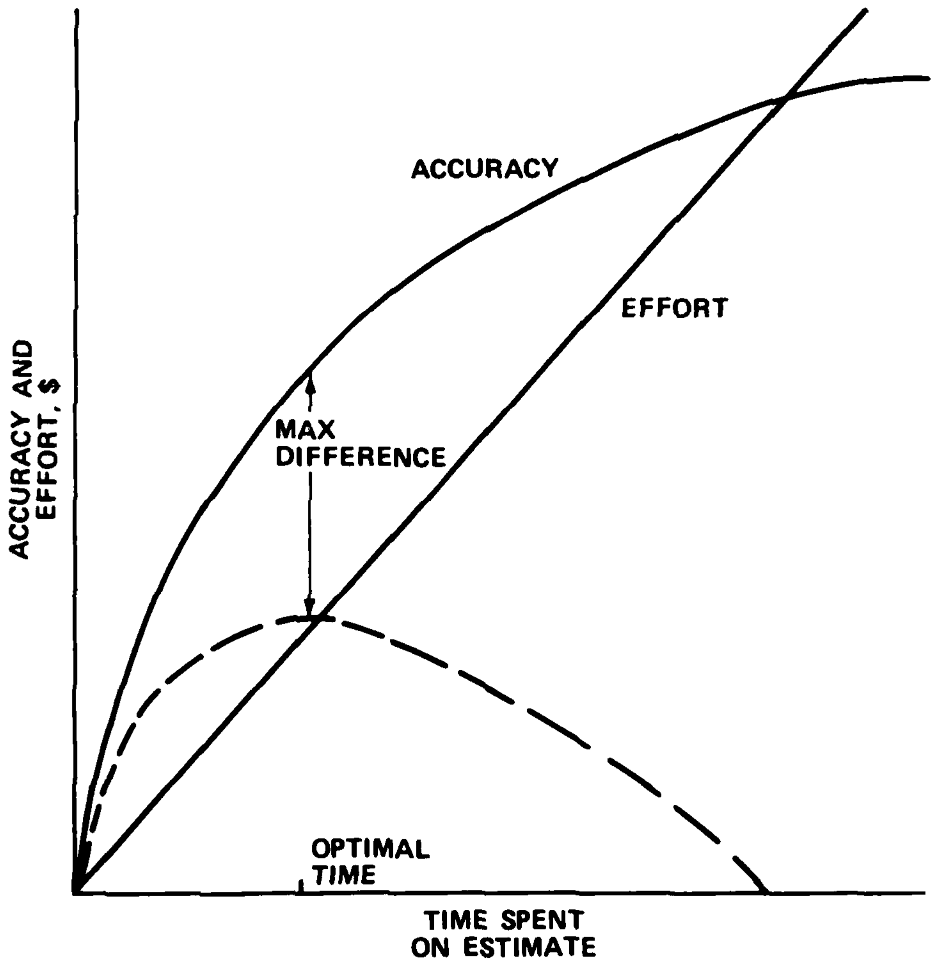

Fig. 5. Selection of Optimal Level of Effort

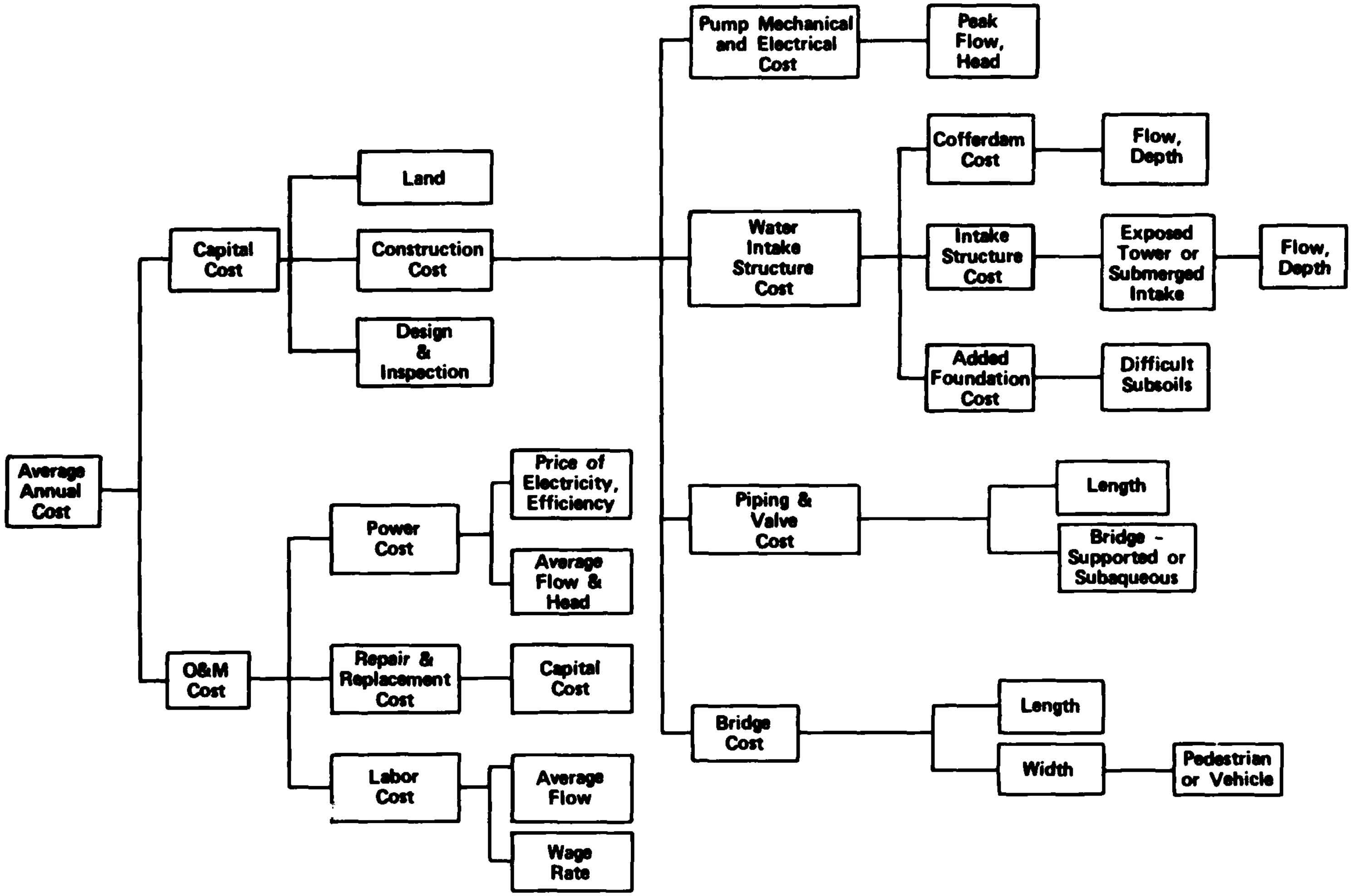

Fig. 6. Diagram Used to Develop Estimating Procedure

structure can be a significant item, yet most planners would not give much consideration to this. By developing the estimating procedure using diagrams like Figure 6, it is easy to identify the input data to the estimating procedure, and to explain to a planner the significance of individual items. The planner is thus aware of the decisions made in the estimating procedure (e.g. no bridge, vehicle bridge, pedestrian bridge).

Limitation to Accuracy. Another problem occurs in cost estimating that does not exist in engineering and physical science--cost estimators must attempt to describe not only physical processes, but human behavior, since the estimator is attempting to model the behavior of bidders on a project. So even though an estimator may have virtually perfect knowledge of the engineering details of a project and current prices, the estimator still cannot be assured of predicting the cost of a project because of the unpredictability of the bidding process. So, even the best estimating procedure may occasionally appear to be inaccurate.

Cost Functions

A cost function is a mathematical relationship between the cost of an item and some parameters (e.g. volume, cost index, local wage rate, horsepower) on which the cost of the item depends.

Some estimators may maintain that instead of using cost functions, one should determine the exact cost. In that case they are using the function

$$\text{Cost} = \text{Constant} \tag{1}$$

a very inflexible formula. Others state that they merely update historic cost. Their cost function is

$$\text{Cost} = \text{Old Cost (Current Index/Old Index)} \tag{2}$$

Equation (2) is based on the assumption that the old item is identical to the current item (a poor assumption). Others would maintain that instead of using cost functions, they actually determine the unit price of the item. This approach reduces to the cost function

$$\text{Cost} = (\text{Quantity}) \times (\text{Unit Price}) \tag{3}$$

This function is useful if the unit price is a constant regardless of the quantity. Otherwise some other relationship is needed to predict unit price as a function of quantity.

The above paragraph illustrates the fact that all estimators use cost functions. However, the functions shown are generally not very useful for planning studies since planning level cost functions need to be applicable over a wide range of sizes and types of facilities and price levels.

Because they must be applicable over the entire range of values for the independent variables, which may be several orders of magnitude, the cost functions can best be described by power functions (i.e. straight lines on log-log

scales). In general the cost function for a facility made up of n items is of the form

$$\text{Cost} = (1 + x) \sum_{i=1}^{n} k_i \, f_i \, a_i \, Q_i^{b_i} \qquad (4)$$

where

Q_i = size of i-th item
a_i, b_i = regression coefficients for i-th item
f_i = present worth or capital recovery factor for i-th item
k_i = ratio of cost indices for i-th item
x = factor for contingencies and minor items
n = number of items

The cost function given in equation (4) is desirable because it is very general, contains factors that can be rationally determined, accounts for all cost items, accounts for variation in unit price for varying quantities purchased, and can be corrected for spatial and temporal variation.

This author developed several guidelines for developing cost functions (Walski, 1980b; Spaine and Walski, 1977) which are summarized below:

1. Power functions (e.g. $C = a\,Q^b$) usually work best especially if data vary over several orders of magnitude;

2. If a cost function has bends and breaks, use piecewise regression rather than a polynomial.

$$\text{e.g.} \quad \text{use } C = \begin{cases} a_1 Q^{b_1}, & Q < Q_o \\ a_2 Q^{b_2}, & Q \geq Q_o \end{cases} \qquad (5)$$

$$\text{instead of } C = \exp\left(a_3 (\log Q)^3 + a_2 (\log Q)^2 + a_1 (\log Q) + a_o\right) \qquad (6)$$

4. If the independent variable is not something a planner will know, make certain that there is a procedure to convert the planner's data into the independent variable (e.g. function relating design flow and pipe diameter);

5. Be certain the index used is relevent (e.g. do not use ENR index to correct power costs);

6. Concentrate on items that contribute significantly to total costs (e.g. do not worry about the cost of the flagpole in front of the treatment plant);

7. Know what is included in the cost data from which the functions are derived (e.g. if some of the data include cost for engineering and design (E&D) and some do not, correct the data so the function either includes E & D or does not). The above guidelines are simple (almost trivial) but failure to adhere to them can result in cost functions of little or no value.

Cost Data

At the heart of any cost function is cost data. These may be costs of entire projects, components of projects or the labor and materials used in the projects, but some type of data must be used to develop the functions. In general there are two types of data: historic data, consisting of cost data for actual facilities along with a description of the facility, and synthetic data, developed by performing detailed cost estimates for a given type of facility based on the sum of the items (e.g. material, labor) required for constructing and operating the facility. At first one would think that historic data would be the best source of data for developing cost functions. However, because no two facilities are ever exactly alike, it is usually better to base planning level cost functions on synthetic data verified with historic data, as discussed in the following paragraphs.

A cost function should describe costs as a function of one or two independent variables which describe most of the variation in the costs. (The effect of price levels can be corrected for after the cost are determined.) For example, the cost of a pump station structure should be given, for a given type of facility as

$$\text{Cost} = f\ (\text{Capacity, Head}) \qquad (7)$$

since both capacity and head affect the size and hence cost of the structure. Ideally one would have historic data for a large number of facilities which differ only in capacity and head. In reality, the structures would have different foundations, different exterior treatments, different area for offices or cleanup facilities, etc. to the point that a substantial portion of the variability in cost is not due to capacity and head. This can be overcome if one has a great deal of data such that the effect of unusually elaborate or spartan structure is lessened, but usually one does not have this luxury.

If synthetic cost functions are used, one can hold all other variables constant while changing only capacity and head. If variation in cost due to other factors is felt to be significant, then these factors can be varied and explicitly considered in the cost function.

Cost functions based on synthetic cost data should not be used until they have been verified against historic cost data. For, while synthetic data are easy to work with, it is possible to overlook or double count a major cost item. This can be discovered in verification.

Caution must be used if data taken from standard cost references (Godfrey; Periera) is used to verify the cost functions. These references do not contain very thorough descriptions of the items, so it is difficult to determine which sub-items are included in an item.

The trend in planning level cost estimating is toward using more synthetic data as evidenced by work of Gumerman, Culp and Hansen (1979), Pound, Crites and Griffes (1975) and Van Note _et al_ (1975).

Estimating as Part of Planning

As mentioned earlier, developing a cost estimate based on detailed plans and specifications is a sequential process in which the designer develops the specifications, and then the estimator prepares the cost estimate. In planning, the estimating procedure is an iterative process. As such the estimator should not be isolated from the planning but rather should be in on decision making from the beginning of a study. One of the primary products of the planning study is the set of estimates for alternatives. The estimator knows the type of data required to perform the estimate and, as such, can guide the planners in collecting the necessary data and developing the designs in sufficient detail to make a good estimate. This will avoid situations in which the estimator is asked the cost of a 10 mgd pump station with no indication given of the type of structure, the head produced by the pumps or whether a wet well will be required. This is an advantage of computer based estimating procedures; the input data required are clearly stated so the planner is aware of the detail of the data required.

Stewart (1981) recognized the relationship between planning and estimating when he stated, "The need for a cost estimate raises virtually every question that must be answered to assure a well-planned work activity... Since good planning and good estimating go hand in hand, cost estimating itself can be used as an excellent planning tool." While Stewart defines planning very narrowly (does not address social, institutional, or environmental aspects), his observation on the relationship between planning and estimating is important.

The cost estimate should not be something tacked on the end of a plan, but an important part of the planning process. For this to occur there must be communication between planners and estimators from the beginning of the study. In some cases, this may consist merely of the planner telling the estimator, "We'd like to use the MAPS computer program. Would you review the cost functions and indicate if we need to apply any correction factors?" Estimators are aware of historic costs and can judge the accuracy of a simplified procedure.

Summary

Development of a planning level cost estimate is a different process than development of estimates based on detailed plans and specifications. Because it is part of an iterative process, planning level estimating requires an approach that yields estimates quickly and reproducibly while accounting for the effects of the most important design parameters. There are tradeoffs between accuracy and level of effort involved in preparing these estimates. As the input data becomes less precise, less effort should be expended in performing the estimate. Some guidelines are presented in this paper for developing cost functions. While the functions can be based on historic or synthetic cost data, it is easier to work with synthetic data. However, the functions must be checked with historic data before they are used. Estimating should not be considered as a process separate from planning.

Planning level cost estimating is evolving from a mysterious art into a rigorous science. As such, planning level cost estimates are becoming more accurate and reproducible and easier to prepare, and thus, of greater value in decision making.

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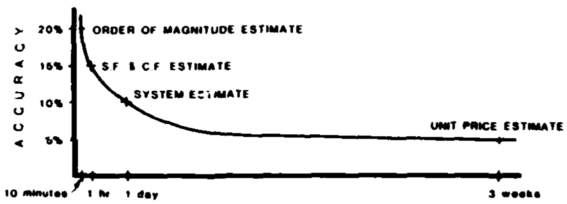
PLANNING WATER SUPPLY PROJECTS: THE SYSTEMS ESTIMATE

When an estimate is made from working drawings, it is reasonable to expect accuracy within 5% of the actual cost of the project. The design has been made for each element, both from an architectural and engineering standpoint. If these contract documents correctly depict this design, the estimator should be able to account for the cost of every item in the project. An estimate involving this level of detail is called the Unit Price Estimate.

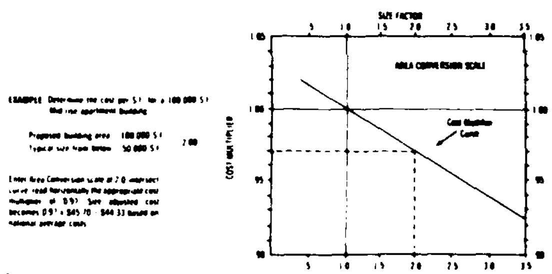


Estimating Time Versus Accuracy

Figure 1

Often in the planning stages, an estimate must be made without the benefit of detailed drawings, having no more than a project outline available. One estimate made from preliminary information is the Order of Magnitude Estimate. The cost of the project is expressed in terms of a usable unit of the facility; in water supply, for example, dollars per gallon. These costs are based on recent costs of similar projects of varying size. This approach offers the least amount of accuracy, no closer than 20% of the actual cost. This estimate does have its place for making cost comparisons and on projects with limited preconstruction budgets.

Another estimate, the Square Foot Estimate, made in the planning stages, is also based on historical data, but a much larger sampling. These projects are listed by type and are standardized for year and location to allow them to be compared by cost and size. The costs per square foot can then be represented for projects having varying space requirements. The median project size, as well as the range of sizes constructed, can be presented. An estimator could evaluate the median project size with respect to his particular project and adjust the area based on quality, complexity, special



Square Foot Base Size

Building Type	Median Cost Per S.F.	Typical Size Gross S.F.	Typical Range Gross S.F.		Building Type	Median Cost Per S.F.	Typical Size Gross S.F.	Typical Range Gross S.F.	
Apartments Low Rise	$ 36.80	21 000	9 700	37 200	Housing (Elderly)	$ 53.40	37 000	21 000	66 000
Apartments Mid Rise	45.70	50 000	32 000	100 000	Housing (Public)	39.10	36 000	14 000	74 000
Apartments High Rise	50.40	310 000	100 000	650 000	Ice Rinks	50.40	29 000	27 000	33 000
Auditoriums	60.10	25 000	7 600	39 000	Jails	104.00	13 700	7 500	28 000
Auto Sales	38.30	20 000	10 000	28 600	Libraries	67.00	12 000	7 000	31 000
Banks	80.70	4 200	2 500	7 500	Medical Clinics	63.60	7 200	4 200	15 700
Churches	57.50	9 000	5 300	13 200	Medical Offices	59.50	6 000	4 000	15 000
Clubs, Country	50.40	6 500	4 500	15 000	Motels	45.50	27 000	15 000	51 000
Clubs, Social	55.30	10 000	6 000	13 500	Nursing Homes	63.30	23 000	15 000	37 000
Clubs, YMCA	56.40	28 300	12 800	39 400	Offices Low Rise	49.60	8 600	4 700	19 000
Colleges (Class)	73.30	50 000	23 500	98 500	Offices Mid Rise	54.80	52 000	31 300	83 100
Colleges (Science Lab)	78.10	45 600	16 600	80 000	Offices High Rise	65.70	260 000	151 000	468 000
College (Student Union)	79.90	33 400	16 000	85 000	Police Stations	88.00	10 500	4 000	19 000
Community Center	57.50	9 400	5 300	16 700	Post Offices	72.60	12 400	6 800	30 000
Convents	57.30	10 000	7 200	16 200	Religious Education	47.10	9 000	6 000	12 000
Court Houses	76.80	32 400	17 800	106 000	Research	77.30	19 000	6 300	45 000
Dept. Stores	35.80	90 000	44 000	122 000	Restaurants	73.30	4 400	2 800	6 000
Dormitories Low Rise	57.30	24 500	13 400	40 000	Retail Stores	35.00	7 200	4 000	17 600
Dormitories Mid Rise	70.40	55 600	36 100	90 000	Elementary Schools	54.50	41 000	24 500	55 000
					Jr. High Schools	53.70	92 000	52 000	119 000
Factories	33.50	26 400	12 900	50 000	High Schools	52.70	101 000	50 500	175 000
Fire Stations	50.60	5 800	4 000	8 700	Vocational Schools	51.70	37 000	20 500	82 000
Fraternity Houses	56.10	12 500	8 200	14 800	Supermarkets	35.00	20 000	12 000	30 000
Funeral Homes	57.30	7 000	4 500	11 000	Swimming Pools	78.00	13 000	7 000	22 000
Garages, Commercial	35.80	9 300	5 000	13 300	Telephone Exchange	80.30	4 500	1 200	10 600
Garage, Municipal	38.90	8 300	4 500	12 600	Terminals, Bus	46.60	11 400	6 300	16 500
Garage, Parking	18.80	163 000	76 400	225 300	Theaters	50.80	10 500	8 000	17 500
Gymnasiums	51.80	19 200	11 600	41 000	Town Halls	54.90	10 800	4 800	23 400
Hospitals	102.80	55 000	27 700	125 000	Warehouses	22.90	25 000	8 000	72 000

Figure 2

conditions, and owner requirements. A cost multiplier would then be applied to the median square foot cost to arrive at the unique cost for that project. This method offers sufficient accuracy, within 15%, to serve as a basis for the decision of whether or not to build.

A much greater degree of accuracy can be obtained at the conceptual design stage with the Systems Estimate. The project could now be examined at a greater level of detail, considering individual portions of the project at a time.

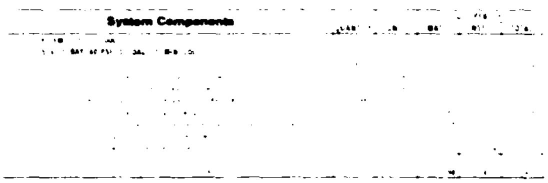


Figure 3

A system can be described as a composite of several unit price components that are interrelated, each expressed in its own unit of measure, that form a major element of a project, expressed in a common unit of measure. The components of a supported slab system, for example, are formwork, reinforcing, concrete, finishing, and curing. A *systems table* is composed of similar systems each having the same components, but differing from each other by some design variable such as dimensions, loading, etc. The supported slab systems, again, are listed in a table by bay size, loading and slab thickness. Different systems tables dealing with the same building element, slabs for example, may be compared for cost.

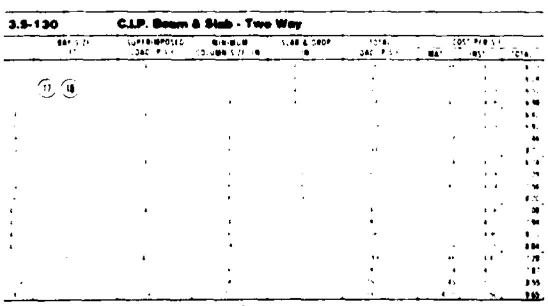


Figure 4

To utilize the systems tables, the estimator must put himself into the role of architect and engineer and "design" the project in its entirety, not at the level of the individual item itself, but from the broader "systems" viewpoint. First, he would collect general project information, if he hasn't already for the Square Foot Estimate, including applicable codes, site investigation (borings), owner's requirements, and design assumptions. In the case of water supply projects, the results of the pilot plant studies would be required.

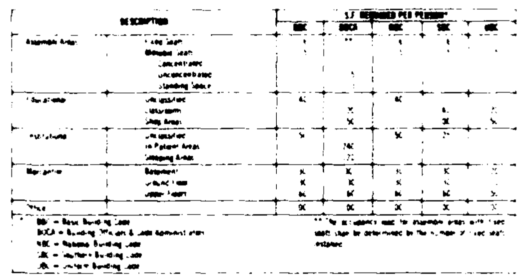


Figure 5

The estimator would now be able to determine the area of the building, using available design aids such as occupancy and building egress requirements, process equipment and tank layout, and special areas such as locker rooms and cafeterias. He would position the building on the site plan to establish the extent and nature of the unbuilt areas; access roads, parking lots, sidewalks, and landscaped areas. Out of this exercise would come the building footprint, the number of floors and the bay size.

At this point it would be advisable to establish an orderly system of bookkeeping, both for general project information (a project summary) and to log selected systems. A numbering system has already been established for systems, called the Uniformat, which contains Divisions 1 through 12, from Foundations through Sitework, respectively. A system should be listed within its division by number and the accompanying information; description, quantity, unit of measure, unit cost and total cost, almost as you would see a line in a Unit Price Estimate.

Now, in order to determine the proper floor loadings, the estimator could consult first the live load requirements by building code, including snow loads and the superimposed dead loads such as floors, partitions, ceiling, electrical and mechanical, as listed in available design aids and reference material. The esti-

Superimposed Dead Load

Component	Load Range (PSF)
Ceiling	5-10
Partitions	10-20
Mechanical	4-8

Figure 6

mator is equipped to select building systems for his estimate. Using bay size and floor loads he can enter the floor systems and make an economical selection. The proper order in which to proceed in this manner through the total building is thus; structure, foundations, closure, mechanical, interiors, and electrical. The approach is always the same--establish the design criteria at the systems level, using design aids, and find a system that will satisfy these criteria.

The criteria for foundations are column load and soil bearing (or pile capacity); for closure, heat loss and aesthetic appearance; for plumbing systems, fixture per capita and gallonage requirements; for heating, loss factor and building volume; for interiors, partition density and fire rating; and for electrical, total watts and available volts.

Of no less importance in the area of water supply planning would be the site work systems. First he would determine the approximate finish grades based on the following information; the hydraulic gradient for gravity flow systems, depth plus freeboard on dams, sidewall depths on tanks and spillways, maximum allowable slopes for cut or fill areas, and, most important, existing finish grades on adjacent properties.

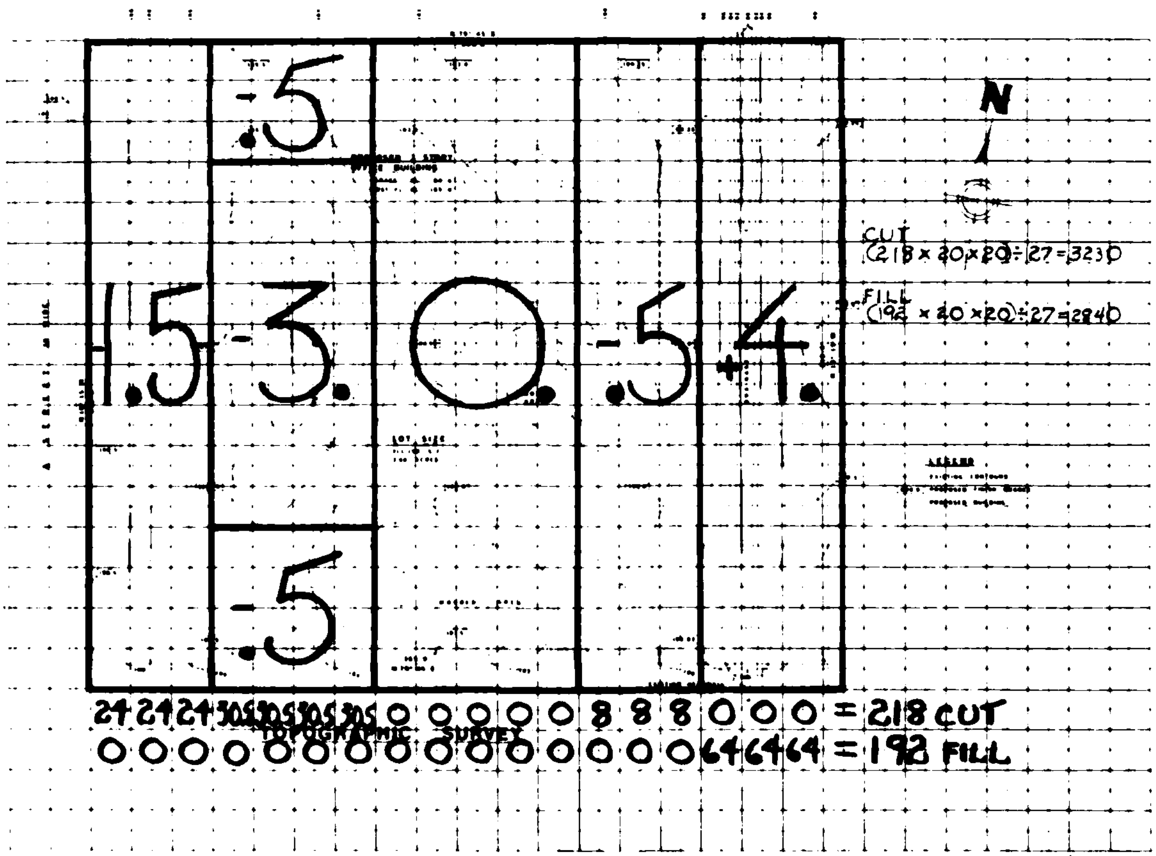


Figure 7 **Excavation-Aid**

Next, based on the site inspection, he would determine the size, density, and extent of trees to be cleared, being careful not to overlook the disposition of the cleared materials.

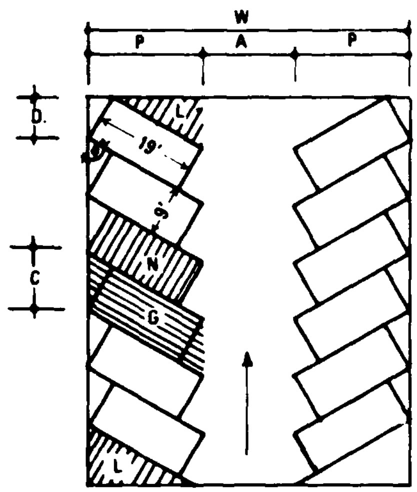
On a plan which contained both the existing and superimposed finish grades, a rapid takeoff of cut and fill materials could be done with a mylar grid overlay sheet.

With the aid of soil borings, the nature of the materials to be cut could be determined for type and swell characteristics. The haul road should be measured for length and grade in both loaded and unloaded conditions to establish round trip durations. The estimator would then select the systems for excavation that match the available equipment and haul conditions.

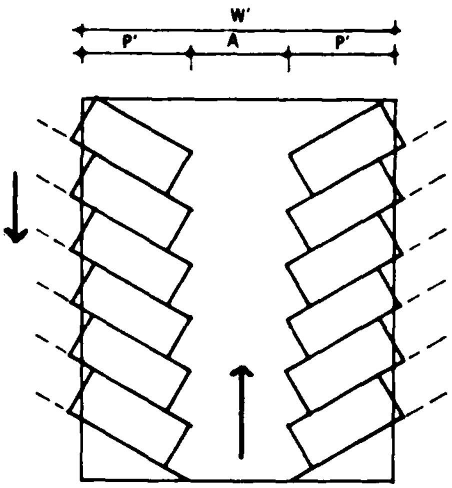
A soils analysis would also be made on available fill material to determine the maximum lifts and equipment passes to achieve desired compaction. All that would remain would be to determine the haul distance from the stockpile to the fill location to allow the estimator to select a suitable system.

The site utility systems could be accounted for by identifying their different characteristics: type utility; overhead, surface or buried; force main or gravity flow. The sizes of the process piping should be available from preliminary design, and any non-process sizes selected from design tables. Source and termination points, intermediate stations, and hydraulic gradients should allow the estimator to lay out the systems on plan and determine trenching requirements, manhole and culvert locations.

Road pavement design criteria require that the weight and number of heavy trucks per day should be estimated. Likewise, the number of heavy vehicles using the parking lot will determine its thickness. Parking area itself is a function of occupancy and use, and varies with angle of the vehicles in the stalls, with respect to traffic lanes.



SINGLE UNIT



OVERLAPPING UNITS

Table 12.5-502 Layout Data Based on 9'x19' Parking Stall Size

Φ Angle of Stall	P Parking Depth	A Aisle Width	C Curb Length	W Width Overall	N Net Car Area	G Gross Car Area	D Distance Lost Car	L Lost Area	P' Parking Depth	W' Width Overall
90°	19'	24'	9'	62'	171 S.F.	171 S.F.	9'	0	19'	62'
60°	21'	18'	10.4'	60'	171 S.F.	217 S.F.	7.8'	205 S.F	18.8'	55.5'
45°	19.8'	13'	12.8'	52.7'	171 S.F.	252 S.F.	6.4'	286 S.F	16.6'	46.2'

NOTE: Square foot per car areas do not include the area of the travel lane.

Retaining structures must be located with any abrupt changes in elevation and take into account the type of earth retained, sandy or clay, and whether it is level, sloped or surcharged. A selection of a retaining system could involve comparing several different wall materials. Less abrupt changes in elevation may produce slopes that still have to be designed for erosion control. Landscaping materials may fit the bill for some of these conditions, but where flowing water is a consideration, as in open channel flow, revetment mattresses must be substituted.

Lastly, landscaping must be considered for the balance of the site areas. The existing site plan and soil borings will determine the amount of loam to be stockpiled. Land clearing itself will create another stockpile of mulching materials. Investigation of the new site plan will determine whether additional loam or mulch will be needed to complete the landscaping operations. Now the estimator is in a position to select the proper lawn and landscaping systems to suit his needs.

None of the criteria necessary to select the appropriate sitework systems should require a lot of time. The estimator only needs to identify the design parameters, not complete the design. The Systems Estimate itself, not including pilot plant studies, soil investigation and preliminary information, should take no more than a day to complete. The touted 10% accuracy (and better by report) is impressive for the time involved, compared with the several weeks for the Unit Price Estimate.

Estimating has been available on the microcomputer for some time now, for the purpose of creating the Unit Price Estimate. The advantage this would have over manually compiling an estimate would be in extending, categorizing, summarizing, and even revising the printed reports that the line item input would generate. The quantity survey still had to be done manually, and the data base had to be created against which the quantity survey could be applied.

Next, the data base containing tens of thousands of line items became available on software. The data base has to be renewed yearly and can be either general in content or developed for a specialized area of construction. It is valuable in eliminating most of the set-up time.

The latest technology has been successful in putting the computer to work on the quantity survey. With the development of the digitizer board on which to lay the drawings and the cursor for scanning, now the microcomputer can even take off quantities. This should reduce the last tedious operation of estimating considerably; and, coupled with a software data base, eliminate manual entry altogether.

For projects in the conceptual stage, software is available that combines the systems with the square foot approach. Sixty-five building models have been compiled in advance, each with its own complement of systems that have satisfied certain design criteria. An estimator need only select the model building he is considering, review the included systems, supply parameters that pertain to his particular project, and the square foot cost for that project will be calculated. Alternate systems may be selected from almost twenty thousand supplied in the data base and substituted into the models. Generating reports, as in the unit price software, is automatic and versatile.

AD P001904

DEVELOPING WATER UTILITY COST ESTIMATES INCORPORATING SPATIAL FACTORS

by Donald L. Schlenger[1]

Introduction

The objective of this paper is to provide the water supply engineer or planner with some perspectives on the spatial aspects of water supply costs. Specifically, it briefly addresses the relationship between capital and operating costs in the transmission and distribution system and the interaction between costs, prices and usage. Finally, it discusses an approach to analyzing operating cost data so as to assess the impact of capital expenditures.

Metropolitan water utilities provide two things. They provide a product: potable water. In delivering that water to the customers' premises, they also provide a service: transportation. The greatest portion of operating and capital costs for a water utility results from providing transportation of water to the customer. The cost of providing this transportation service has been growing at an alarming rate in recent years due to rising energy costs. For example, Figure 1 shows how the costs of electric power rose over three years at one booster pumping station. As a result, the differences from one location to another in the distribution system in the cost of providing water have become more acute.

Cost estimates that recognize these spatial differences are important for:

- Long run capital planning of the transmission and distribution system. For example, they can be used to determine whether the transmission facilities between two points are too energy intensive because pipe diameters are too small, or whether an additional booster pumping station should be built.
- Energy management programs to minimize operating costs while maintaining adequate pressure throughout the distribution system.
- Rate setting when rates can vary from one location to another (for example, from the city to the surrounding county). Spatially differentiated cost estimates can provide a rational basis for allocating the costs of facilities that are shared by customers.

Cost estimates for water supply planning should be developed and used with an understanding of how they impact on management decisions and policies, customers' response, and system operation. Spatial variations in these costs may be reflected in spatial variations in their impact.

Spatial Variations in Water Supply Costs

Most large urban water supply systems rely primarily on surface water supplies (American Water Works Association, 1978). For these systems, it tends to be costlier to serve more remote locations for several reasons. While the sources of raw water may not be located near the urban center, principal treatment and distribution facilities often are because metropolitan areas tend to grow more or less radially (Chinitz, 1964)

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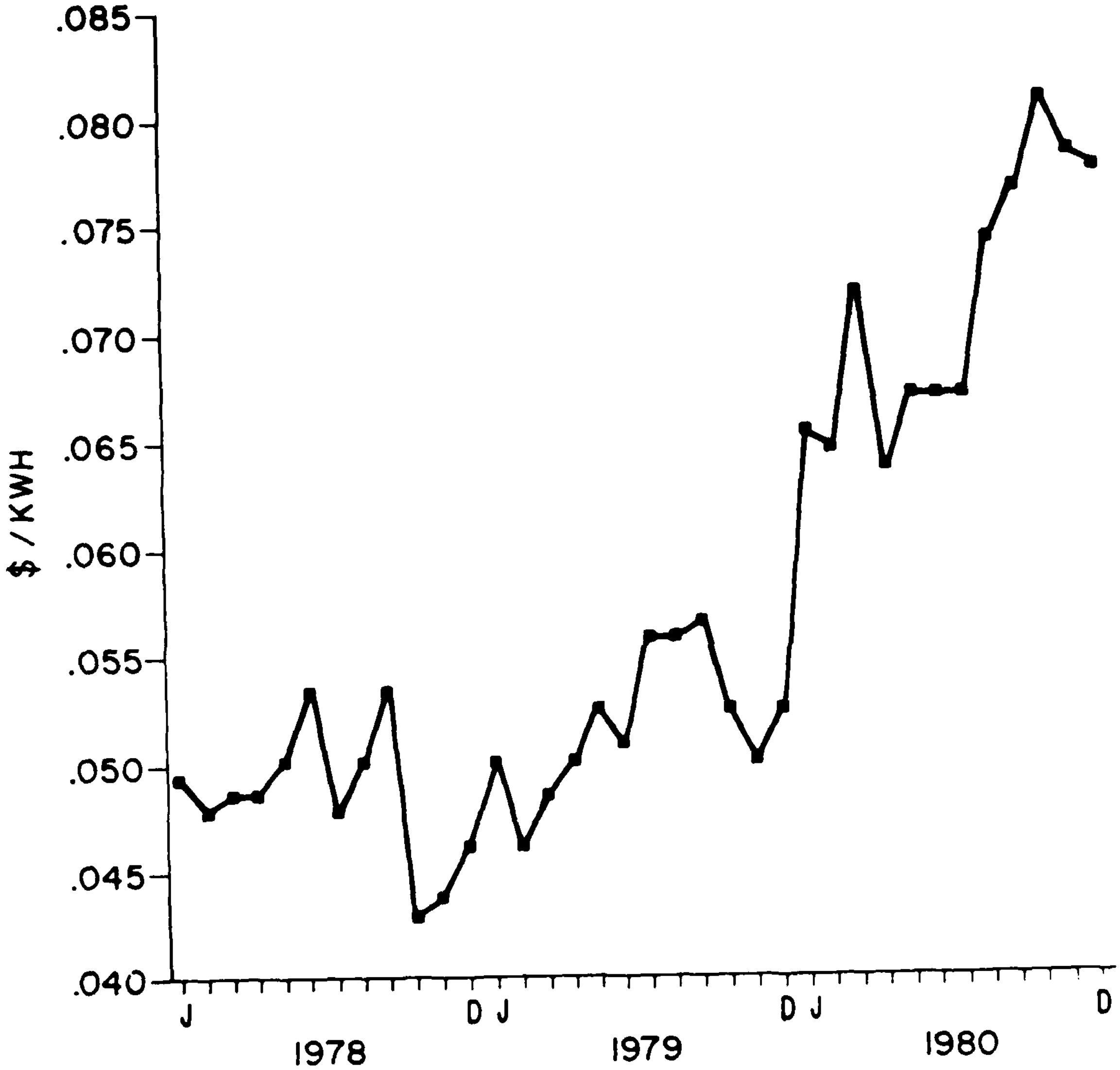

Figure 1. Cost of electric power for a 23 mgd pumping station, 1978-1980.

and because it is usually far more economical to build and operate one large central treatment plant (and the distribution system around it) than several smaller ones.

The cost of delivering water through such systems increases with distance from the treatment facilities due to the energy costs for pumping and the additional capital facilities required. The energy used for pumping serves to (1) raise the water to customers living at higher elevations than the treatment plant, (2) provide adequate service pressure to customers (usually accomplished by raising the water to additional elevation), and (3) overcome the friction resulting from forcing the water through the pipes of the transmission and distribution system. The energy required per unit of time to raise the water in elevation is simply the product of the height to which it must be raised, known as the static head, and the rate of flow. The energy required to force water through a pipe is a function of the diameter of the pipe, its length and roughness, and the rate of flow. This fundamental relationship is commonly expressed by the Hazen-Williams formula for flows in circular conduits under pressure (Fair, Geyer and Okun, 1971), which may be written as

$$(1) \qquad h_f = k\, l\, (x/C)^{1.85}\, D^{-4.87}$$

In this equation, h_f is the friction or dynamic head loss, l is the length of the pipe, D is the diameter of the pipe, x is the rate of flow through the pipe, C is a coefficient of roughness of the pipe, and k is a constant. Friction head loss typically has units of feet. As with the static head, the total energy required per unit of time to overcome the friction head is simply the product of the head loss and the rate of flow. The total head loss to be overcome by pumping is the sum of the static and friction head losses.

From this equation it can be seen that the greater the distance l water has to be pumped through a pipe, the greater the cost per gallon. This relationship is linear and is shown in Figure 2. Also, the greater the rate of flow x in the pipe, the greater the cost per gallon, and this cost rises at an increasing rate, as illustrated in Figure 3. While there is usually an intricate grid of interconnecting pipes between a pumping station and any point in a large distribution system supplied by it, McPherson (1960) has shown that between them the friction head loss can be represented by Figures 2 and 3 and by a relationship of the form $h_f=kx^m$, where x is the total flow in the system and m varies between 1.86 and 2.0. This is similar to Eq. 1.

In addition to the increased pumping costs, when water must be transported to greater distances, more facilities (pipes, valves, etc.) are required. For larger water systems, it is too costly to provide all of the pumping at the principal treatment and distribution works. Instead, the system is divided into pumping districts characterized by independent distribution piping networks, and booster pumping stations are used to pump the water from one pumping district to the next. With these booster pumping stations are usually associated both ground and elevated tanks or reservoirs. Typically, a ground reservoir is filled by water pumped into a lower portion of the distribution system. The booster pumping station draws water from this reservoir and discharges it, often into an elevated tank, at a pressure sufficient to serve the customers in the higher pumping district. The facilities associated with booster pumping stations compound the degree to which an increase in distance served is marked by an increase in capital investment. Larger water supply systems generally have several pressure districts.

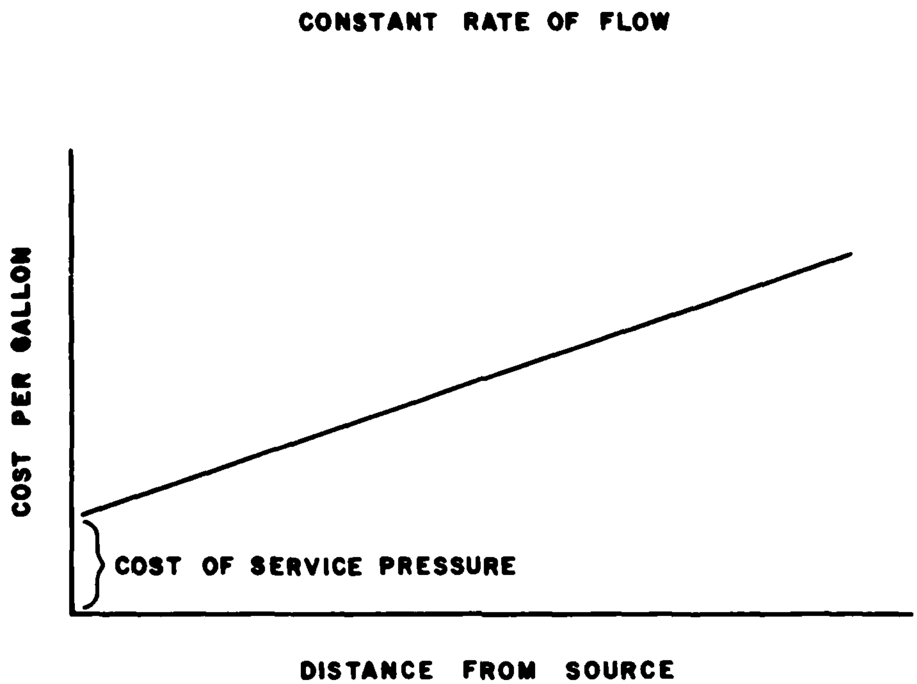

Figure 2. Pumping cost as a function of distance.

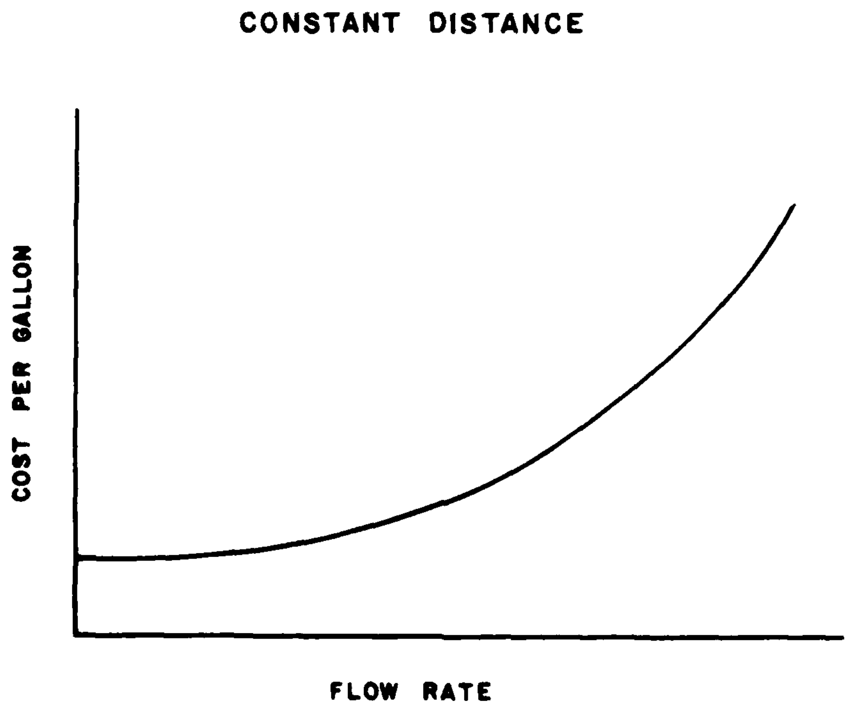

Figure 3. Pumping cost as a function of rate of flow.

Interactions of Capital and Operating Costs

New water supply system capital investment is undertaken to meet current or expected growth among existing customers, to meet growing needs among new customers in new areas, to replace worn out plant, or to improve efficiency and reliability. The size, type, location, condition and costs of operating current facilities affect decisions about the size, type, location and timing of new capital investments. These investments in turn affect future operating costs and future investment decisions.

These relationships may be illustrated by considering the simple example of the water supply system shown in Figure 4. In this example a central treatment plant delivers water into the transmission and distribution system in pumping district 1. Booster pumping stations for pumping districts 2 and 3 draw water from service reservoirs filled from pumping district 1 and deliver it to elevated tanks that provide the storage and service pressure required to serve these pumping districts. A similar arrangement holds for pumping district 4.

This water supply system may be represented conceptually by a set of nodes and links, as shown in Figure 5. The nodes are centroids of demand within each pumping district, and the demand at the node is simply the sum of the demands by customers throughout the pumping district. At each node, water may be either consumed or passed on to the next node. The links represent the combination of transmission and distribution mains that collectively carry water from one node to the next. Normally there are only a few major mains between nodes, and these may be combined into one by the method of equivalent pipes (Fair, Geyer and Okun, 1971). The friction head loss between the two nodes would be represented by Eq. 1 rewritten as follows:

(2) $$h_f^{0.54} = k \times ED^{-2.63}$$

where ED is the effective diameter of the link between the two nodes, x is the flow between them, and k now includes the roughness coefficient and the length l. Writing Eq. 2 in terms of x for the existing link diameter (referred to as D_1) gives

(3) $$x = h_f^{0.54} k^{-1} D_1^{2.63}$$

Suppose it is decided to expand the capacity of the link between the two nodes. Of the several ways to do this, the most common is to build an additional, more or less parallel, transmission main. Assume this transmission main has diameter D_2. The head loss h_f is the same for both mains since they connect the same nodes. Writing Eq. 3 for two mains of equal length and roughness having diameters D_1 and D_2 and passing flows x_1 and x_1 gives

(4) $$x_1 = h_f^{0.54} k^{-1} D_1^{2.63}$$

(5) $$x_2 = h_f^{0.54} k^{-1} D_2^{2.63}$$

Since the total flow in both mains x equals x_1 plus x_2, Eqs. 4 and 5 can be combined to give

$$(6) \qquad x = h_f^{0.54}\ k^{-1}\ (D_1^{2.63} + D_2^{2.63}),$$

Substituting this expression back into Eq. 2 yields

$$(7) \qquad ED = (D_1^{2.63} + D_2^{2.63})\ 0.38$$

where ED is the effective diameter of the combined mains.

Mains of different lengths or roughness coefficients can be accommodated by using the Hazen-Williams equation to compute an equivalent diameter for one main if its length and roughness coefficient were assumed to be the same as the other main. This "new" diameter could then be used in Eq. 7. Eq. 7 may be easily generalized to more than two mains. Eqs. 2 and 7 together illustrate how a capacity expansion impacts on distribution system operating costs required to overcome friction head loss and how existing capacity affects the impact. More sophisticated and detailed approaches to these interrelationships for transmission mains may be found in the literature. For example, Deb has developed formulas for optimal branched pipe networks (1974) and pumping systems (1978).

To illustrate the relationships of Eqs. 2 and 7, consider the following numerical example. Suppose a water distribution system contains a booster pumping station that delivers water through a set of mains with a combined effective diameter of 20 inches to an elevated tank that is 230 feet higher in elevation and 10,000 feet away. Assume the roughness coefficient for this link is 100. The friction head loss as a function of the flow through this link is given by the curve in Figure 6 labeled D=0". If an additional transmission main of diameter D_2 is constructed on this link, the effective diameter of the link will be given by Eq. 7. The friction head loss as a function of flow is shown for various sizes of the additional main by the other curves in Figure 6. For example, if the new main is 16 inches in diameter, the new effective diameter would be 23.7 inches. With the additional main, the friction head loss at 3.0 million cubic feet per day would be reduced from 550 feet to 250 feet.

In addition to overcoming the friction head loss, the booster pumping station must provide a static lift of 230 feet. Assume that electricity costs \$0.05/kwh and the pump station efficiency is 0.85. Based on these numbers, the total pumping cost in thousands of dollars per day as a function of flow for different sizes of the additional main is plotted in Figure 7. The greater the flow, the higher the daily pumping cost. At a flow of 3.0 million cubic feet per day, adding a new main of diameter 16 inches reduces the cost from \$3,250/day to \$1,940/day. How the operating cost per day is reduced as the diameter of the additional main is increased is shown in Figure 8 for three different flows. As the diameter is increased, the cost of overcoming friction head loss becomes smaller in relation to the cost of overcoming static head. Figure 8 shows the tradeoff between capital investment and operating costs.

To examine this relationship in more detail, capital costs can be considered explicitly. The total capital cost of a pipeline including installation can be expressed as a function of diameter:

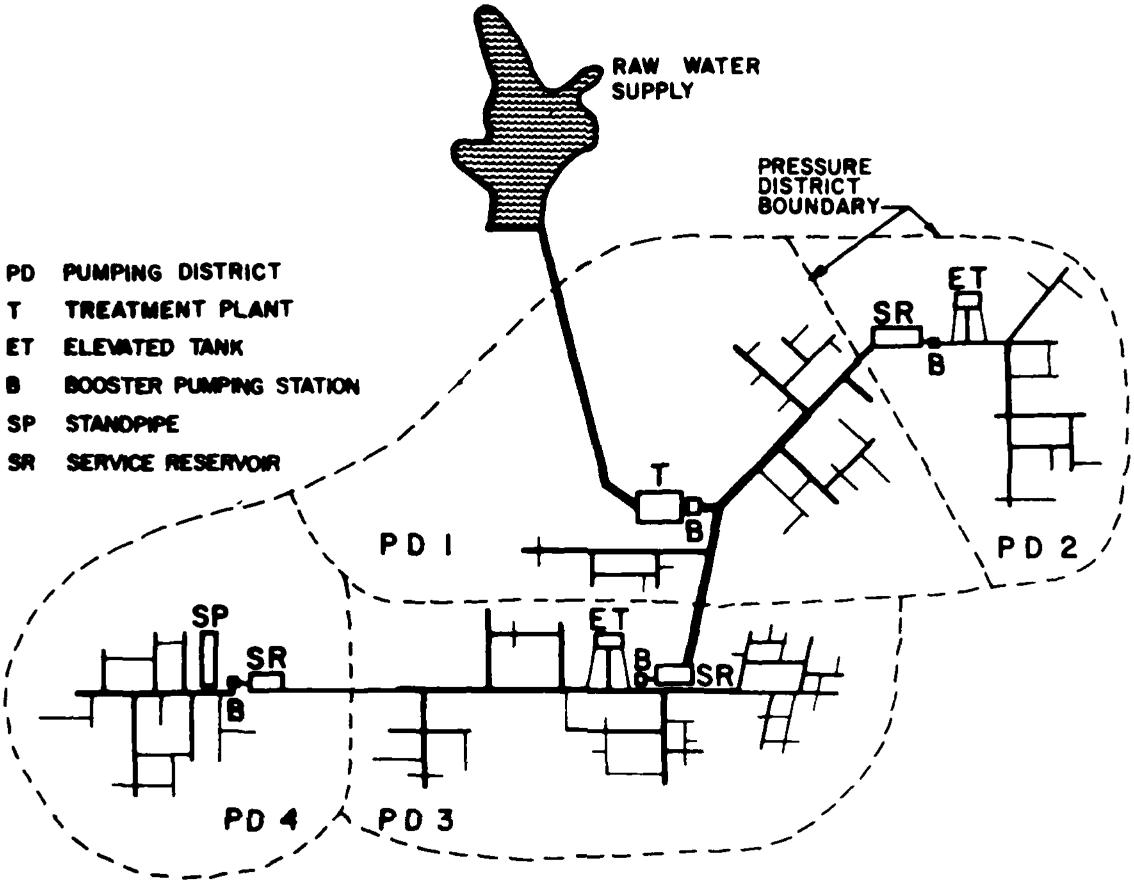

Figure 4. Typical metropolitan water system.

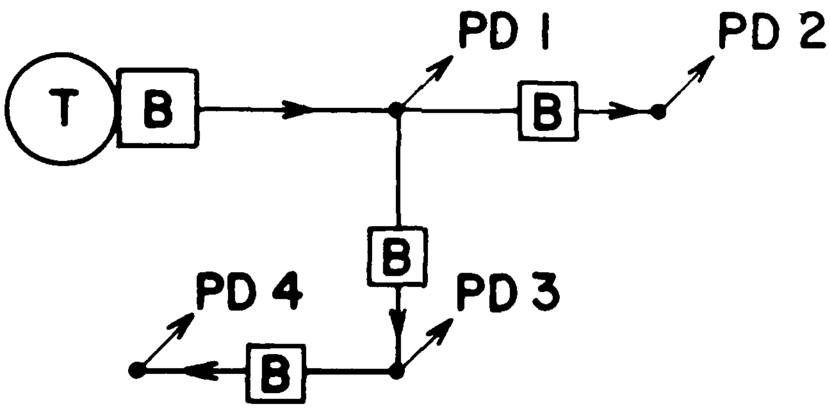

Figure 5. Conceptual representation of typical water system.

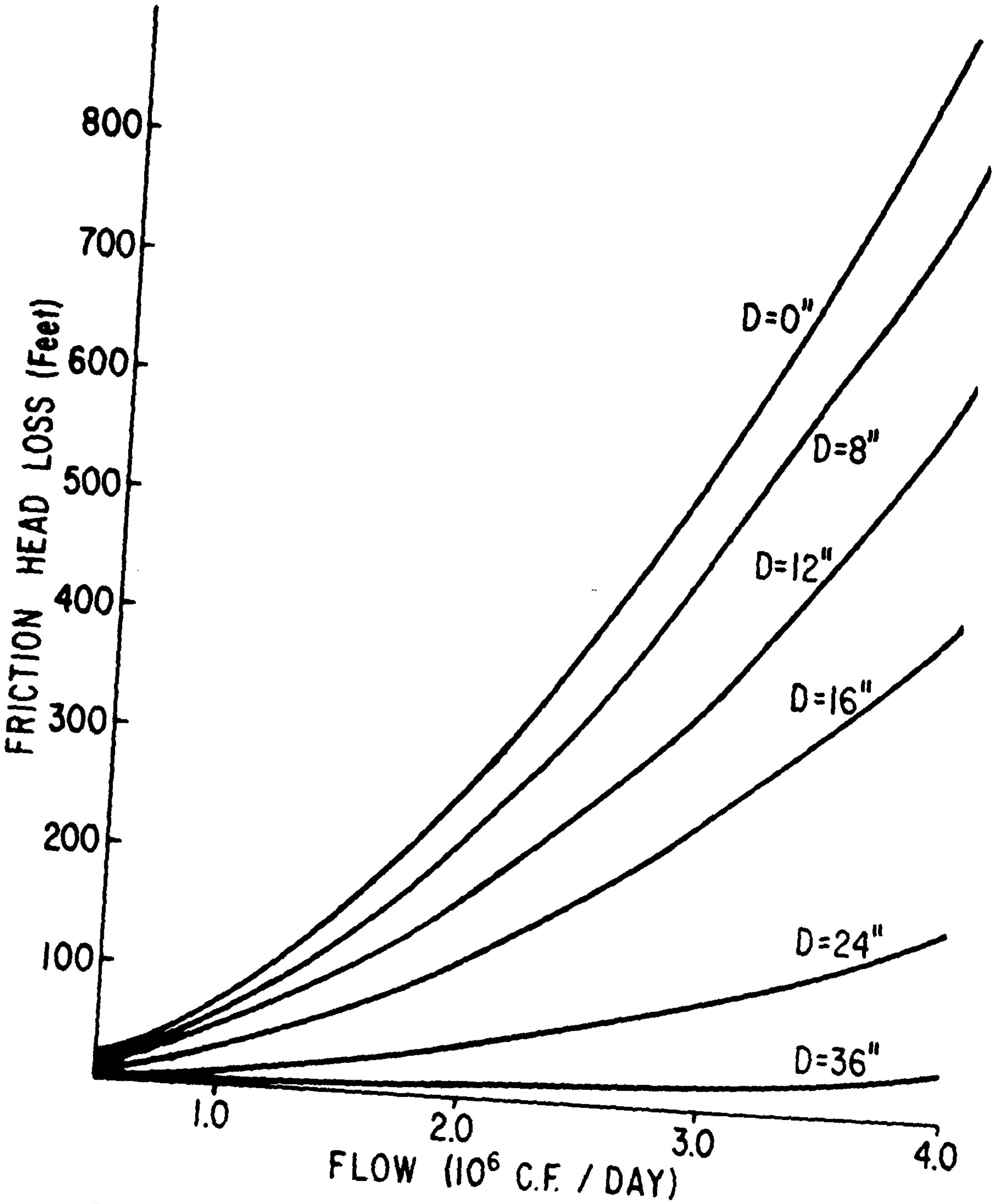

Figure 6. Friction head loss versus flow for link expansion.

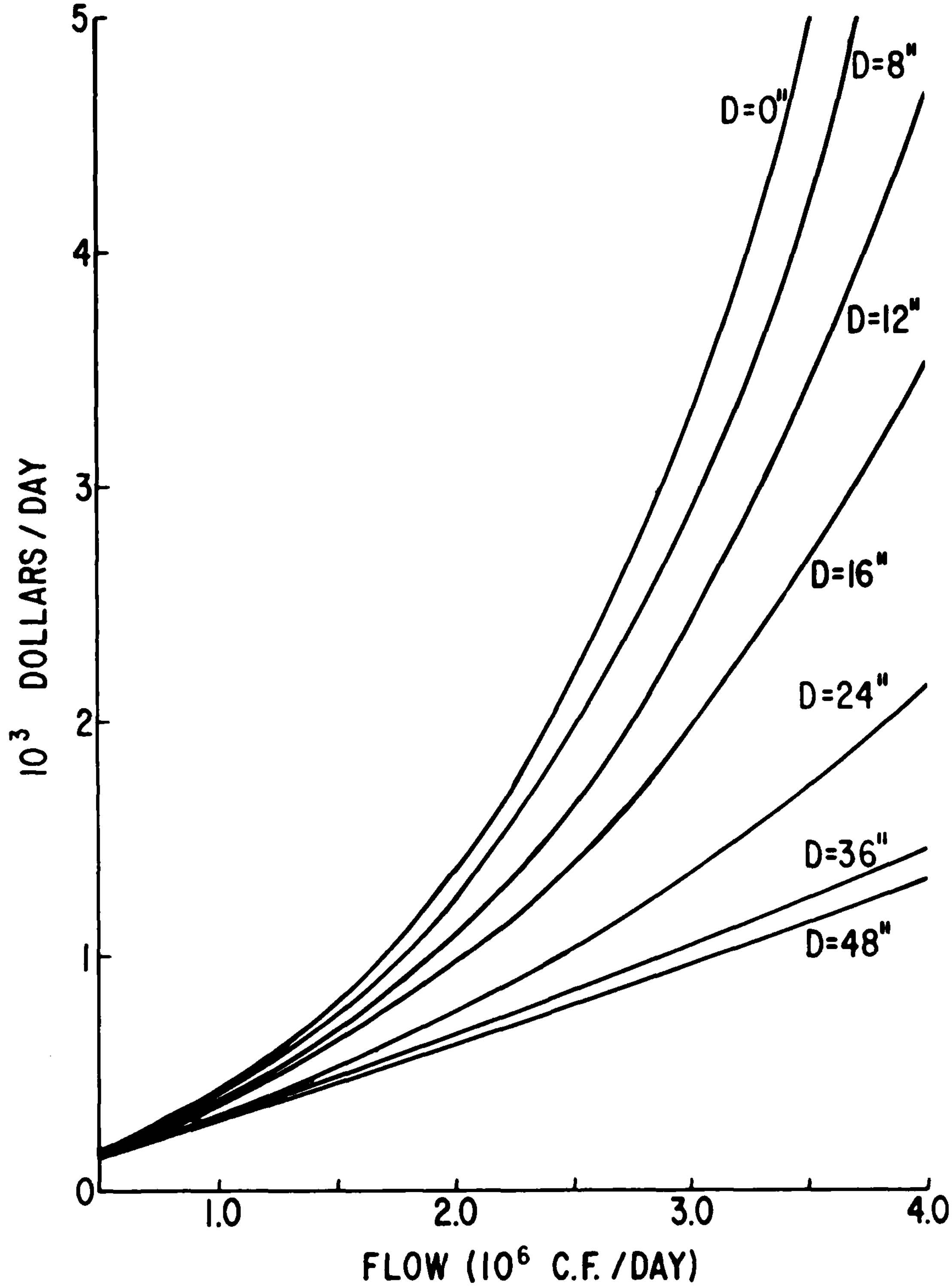

Figure 7. Pumping cost versus flow for link expansion.

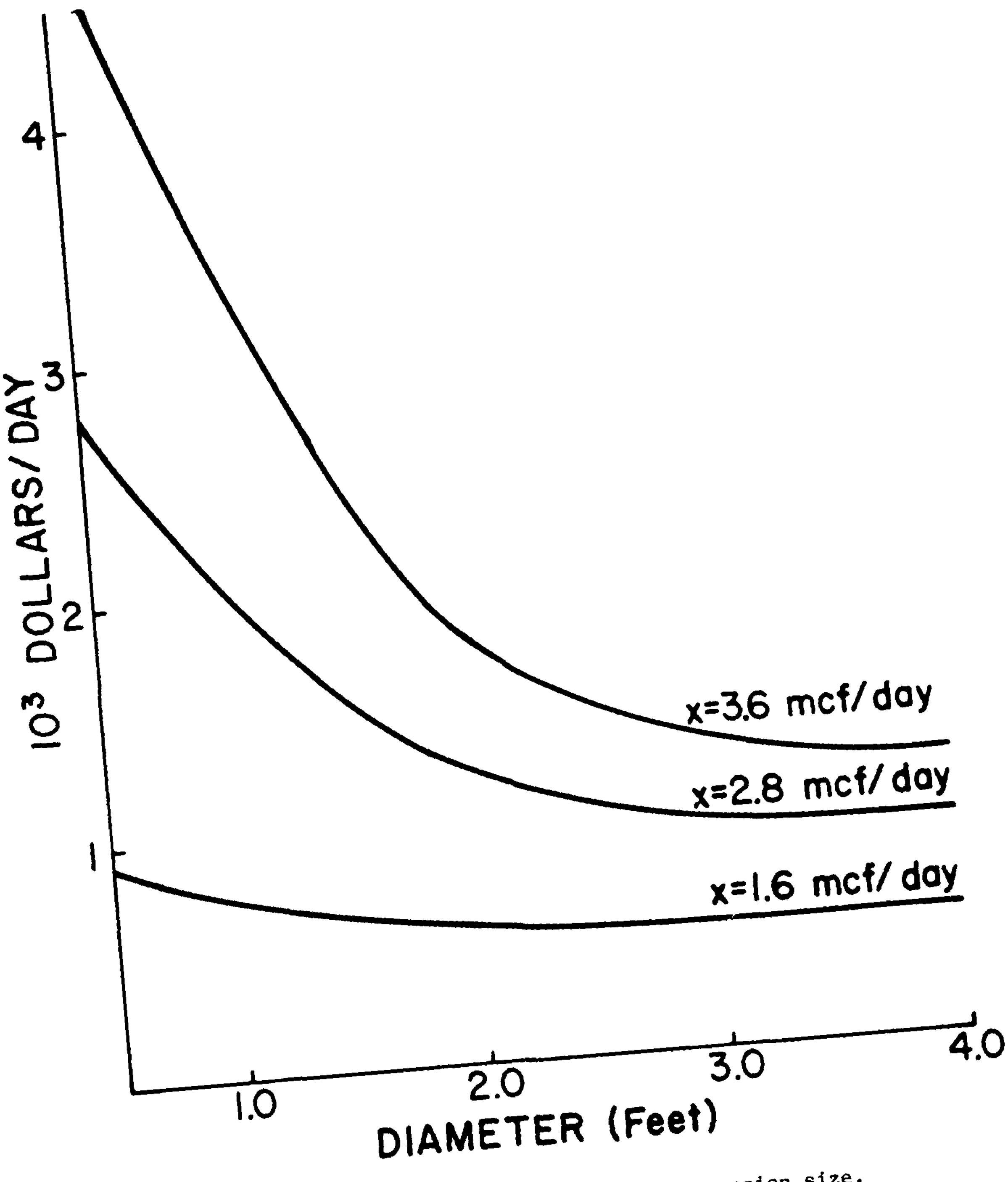

Figure 8. Pumping cost versus link expansion size.

(8) $$C_m = l\ a\ D^b,$$

where l is the length of the pipeline, D is the diameter, a is a coefficient, and b is an exponent. Typical values in 1982 dollars are a=38.98 and b=1.29 for l and D in feet (Deb, 1978).

The capital cost of a pump may be expressed as a function of flow and total head:

(9) $$C_p = u\ h^v\ x^w,$$

where h is the total head, x is the flow rate, u is a coefficient and v and w are exponents. Typical 1982 values are u=$1,232, v=0.642 and w=0.453 (Deb, 1978).

The objective is to minimize the present value of capital and operating costs. Alternatively expressed for the example, it is to maximize the present value of operating cost savings as a result of expanding the link less the capital costs of the expansion. Assume that the expansion results in a perpetual stream of operating cost savings, and that the discount rate for these savings is 7% per year. Then the present value of the savings is $365(.07)^{-1}c'$, where c' is the cost saving per day. Plots of the net present value savings as a function of the diameter of the additional main for different flow rates are shown in Figure 9. From the diagram, it can be seen that there is an optimal sized expansion for any flow rate above about 1.4 million cubic feet per day. Below this rate of flow, it is not worthwhile building the expansion. It may also be noted that the net present value cost savings is relatively insensitive to changes in the diameter of the expansion near the optimal size.

User-Sensitive Interactions

The above analytical approach would be adequate were it not for the interaction between costs, prices and demands (Figure 10) and the changes in these factors over time. In particular, the costs of building and operating water supply systems, including the administrative and financial costs, must be somehow recovered, usually through some combination of tariffs and taxes paid by the users. To the extent that these costs are included in the water bill (and especially commodity charges) customers will pay directly for what they use.

It has been empirically demonstrated, and it is universally acknowledged, that water customers respond in some measure to changes in the price of water by adjusting their rates of consumption. This phenomena is known as the price elasticity of demand. If, for example, the price of water increases by 10% and customers therefore reduce their consumption by 5%, the aggregate price elasticity of demand is said to be -0.5. Price elasticity of demand depends on myriad factors, among them the current price and level of consumption, the consumers' income levels, and the mix of applications to which the water is put. Published estimates of price elasticities of demand for water supply range from as little as -0.08 for indoor residential use (Carver, 1978) to as much as -1.57 for outdoor sprinkling demand in well-watered climes (Howe and Linaweaver, 1967). Typical estimates fall in the range -0.10 to -0.50.

Following Figure 10, if a capital expenditure is undertaken so as to meet anticipated demands or service requirements for water and this requires a significant

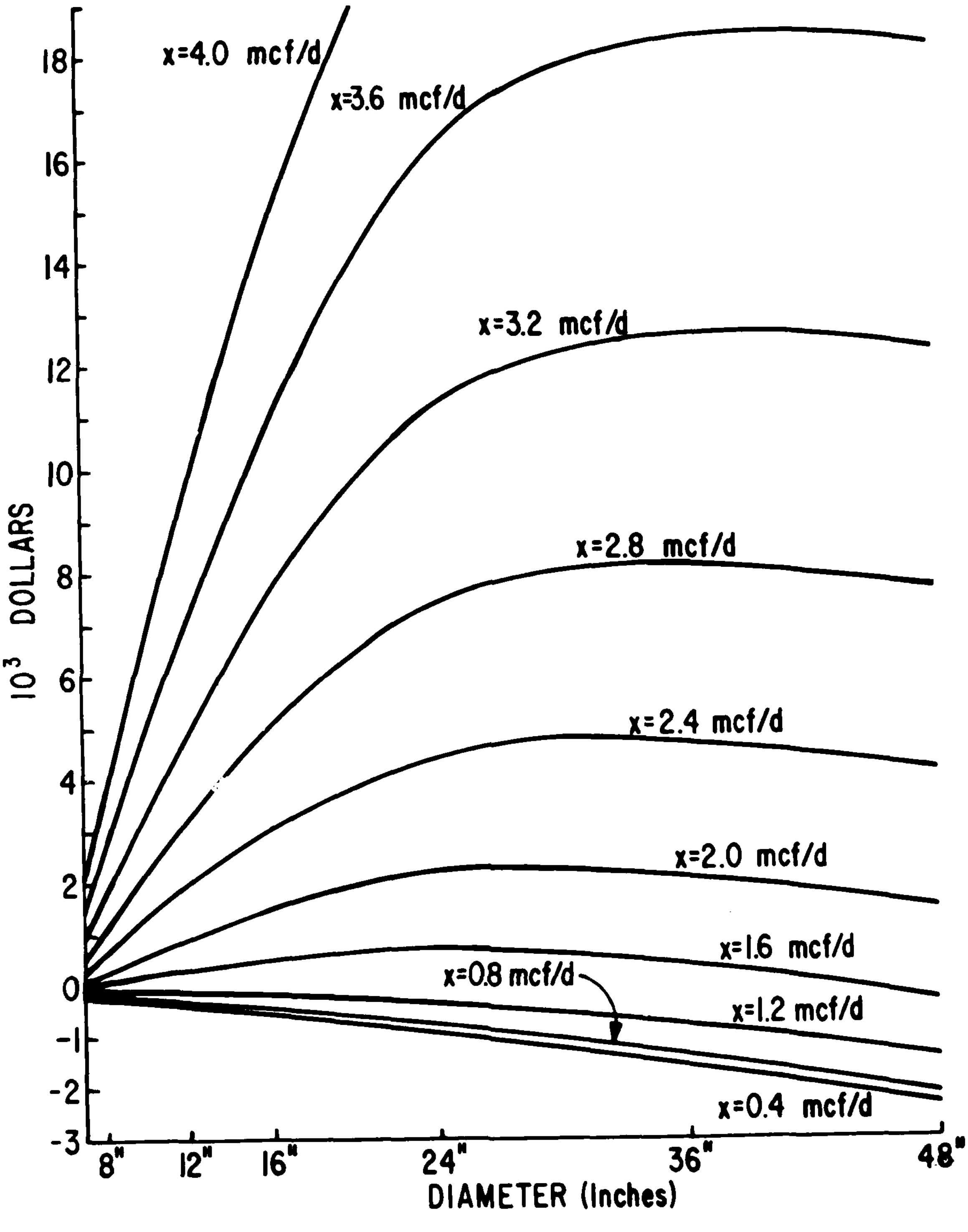

Figure 9. Net present value cost savings versus link expansion size.

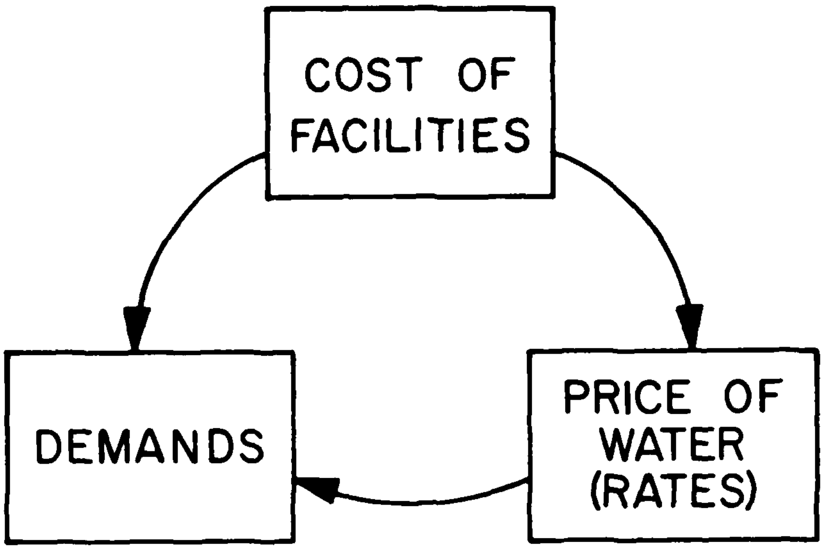

Figure 10. Interaction of costs, prices and demands.

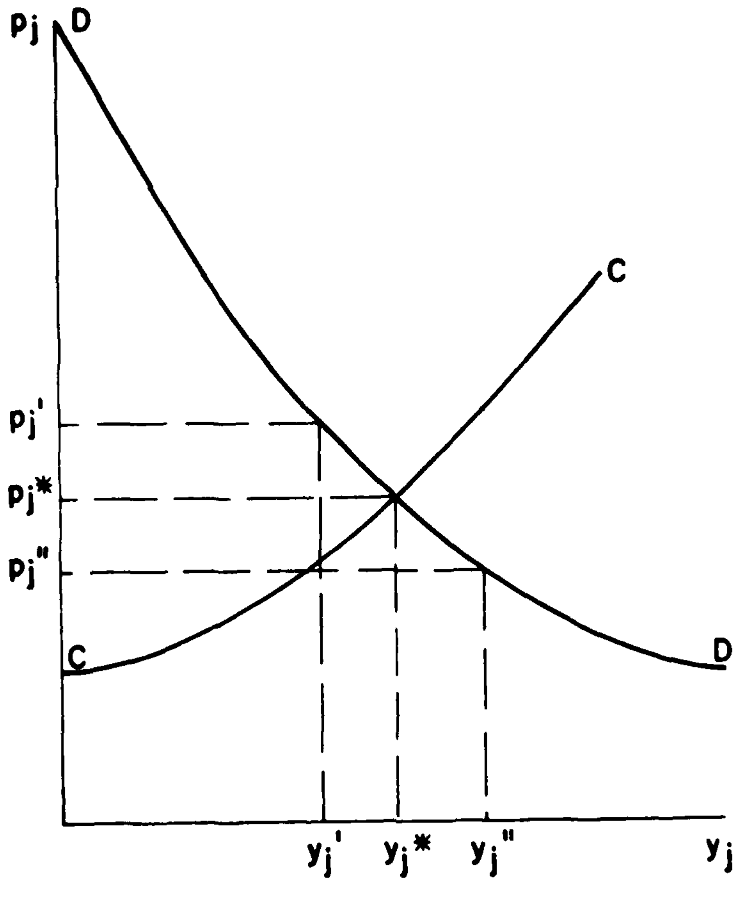

Figure 11. Simple supply-demand relationship.

increase in rates to recover the costs, the very demands that the project was designed to meet may be curtailed as customers respond to the increased prices. The result is that the project would be overdesigned. If consumption is less than anticipated, the price will have to be raised further to obtain sufficient revenues to cover costs, creating additional adjustments.

In all but the most extreme cases, this adjustment process would eventually stabilize. However, adjustments take time, the size of capital facilities once built is not easily altered, and patterns of water demand are continually changing. Because of these factors, most water supply planners and engineers throw up their hands and assume that projected water demands are actually non-varying water requirements. Where customer demands are growing and their response to prices is somewhat elastic, this "requirements" approach can lead to economic inefficiencies and distortions. If demands are growing at different rates throughout the distribution system, if they have different elasticities, and if the costs of providing service differ significantly from one place to another, failure to consider spatial differences can aggravate the problem.

Spatial Pricing to Reflect Cost Differences

When there are significant differences in the cost of serving customers from one portion of the water supply system to another and these costs differentials are not matched by differentiated rates, inefficiencies in the development and use of water supply facilities are likely. These inefficiencies affect short-run operating decisions and long run investment decisions of both the utility and its customers.

For instance, in the short run customers who live close to the treatment plant and under uniform pricing rules are charged more than what it costs (in terms of real resources) to deliver water to them likely use too little. Society would be better off if they used more, since the benefit they gain from the additional consumption is greater than the additional costs of providing it to them. For example, the input mix chosen by industrial customers close to the treatment facilities may be too water deficient relative to that mix which would produce the same product at less cost from society's point of view. Conversely, customers who require extensive distribution facilities to supply water to them and who under uniform pricing pay less than what it costs to deliver water to them likely use too much. For example, the suburban residential customer whose water is underpriced may use the garden hose to clean leaves off his driveway; social welfare would be better served if he used a broom.

In the long run, the investment decisions made by customers of the utility whose prices do not reflect costs will also be distorted and inefficient. For example, suburban users who water is underpriced may develop lawns and landscaping which require more water than they would use were prices to truly reflect costs. Developers who do not bear the full cost of extending the water supply system to their subdivisions may build too much or at too low a density.

An approach to planning that takes account of these factors is needed. This approach will be defined in terms of pumping districts, since large cost differences most often occur at pumping district boundaries.

The demand for water by the customers in each pumping district is a function of the price. Their demand functions can be combined to form a single aggregate demand function for that pumping district which may be expressed as

(10) $$y_j = y_j(p_j)$$

where y_j is the quantity demanded per unit time period in pumping district j and p_j is the price of water charged there. The demand function is represented by the curve D-D in Figure 11, and is assumed to have a negative slope and be uniquely valued, continuous and differentiable in the area of interest.

Since the quantity demanded is variable, and there is an interaction between costs, prices and demands, the objective cannot be simply to minimize the costs of water supply. Rather, it should be to maximize total benefits less total costs. Total benefits are usually represented by the area under the demand curve from 0 up to the quantity demanded y_j:

(11) $$TB_j = \int_0^{y_j} p_j(y)dy$$

where $p_j(y)$ is the inverse demand function and y is a dummy variable of integration, also with units of flow.

Suppose that for any quantity of water consumed per unit time period y_j, the marginal cost of providing the next unit of water is represented by the curve C-C, which is the first-order differential of the total cost curve. To maximize total benefits less total costs, the proper price to charge is p_j* and the proper quantity to produce is y_j*. If the price charged was p_j' (and the corresponding quantity consumed was then y_j'), the production and consumption of an additional unit would result in incremental benefits greater than incremental costs, so it would be worthwhile. Conversely, if the price and quantity were respectively p_j" and y_j", the production of an additional unit would result in incremental costs greater than incremental benefits, so it would not be worthwhile.

This principle of marginal cost pricing is illustrated in the spatial context by Figure 12, taken from Turvey (1968). Suppose the customers in two pumping districts have aggregate demand functions D_1-D_1 and D_2-D_2, respectively. (That D_2-D_2 is shown to be greater than D_1-D_1 is of no import to this example; they could be reversed.) Suppose that the long run marginal costs of serving each group of customers are constant at $LRMC_1$ and $LRMC_2$, respectively. If a uniform price p is charged, customers in pumping district 1 will purchase $0x_1$ and customers in pumping district 2 will purchase $0x_2$. However, if the price charged each group of customers reflected marginal costs, customers in pumping district 2 would consume only $0x_2$', while customers in pumping district 1 would expand their consumption to $0x_1$'. The incremental cost of expanding the volume of water supplied to the pumping district 1 customers is represented by the area A, while the incremental benefits from providing the additional water to these customers is the area A+B. The costs saved by serving the customers in pumping district 2 less water is the area C+D+E+F, while loss of benefits to those customers is represented by the area C+D+E. Therefore, the total net efficiency gain associated with the differentiated pricing over uniform pricing is represented by the sum of the areas B and F.

In addition to the interactions when demand varies with price, the impact of costs is further complicated by the joint cost nature of water supply systems, since customers in different parts of the distribution system use various facilities in common

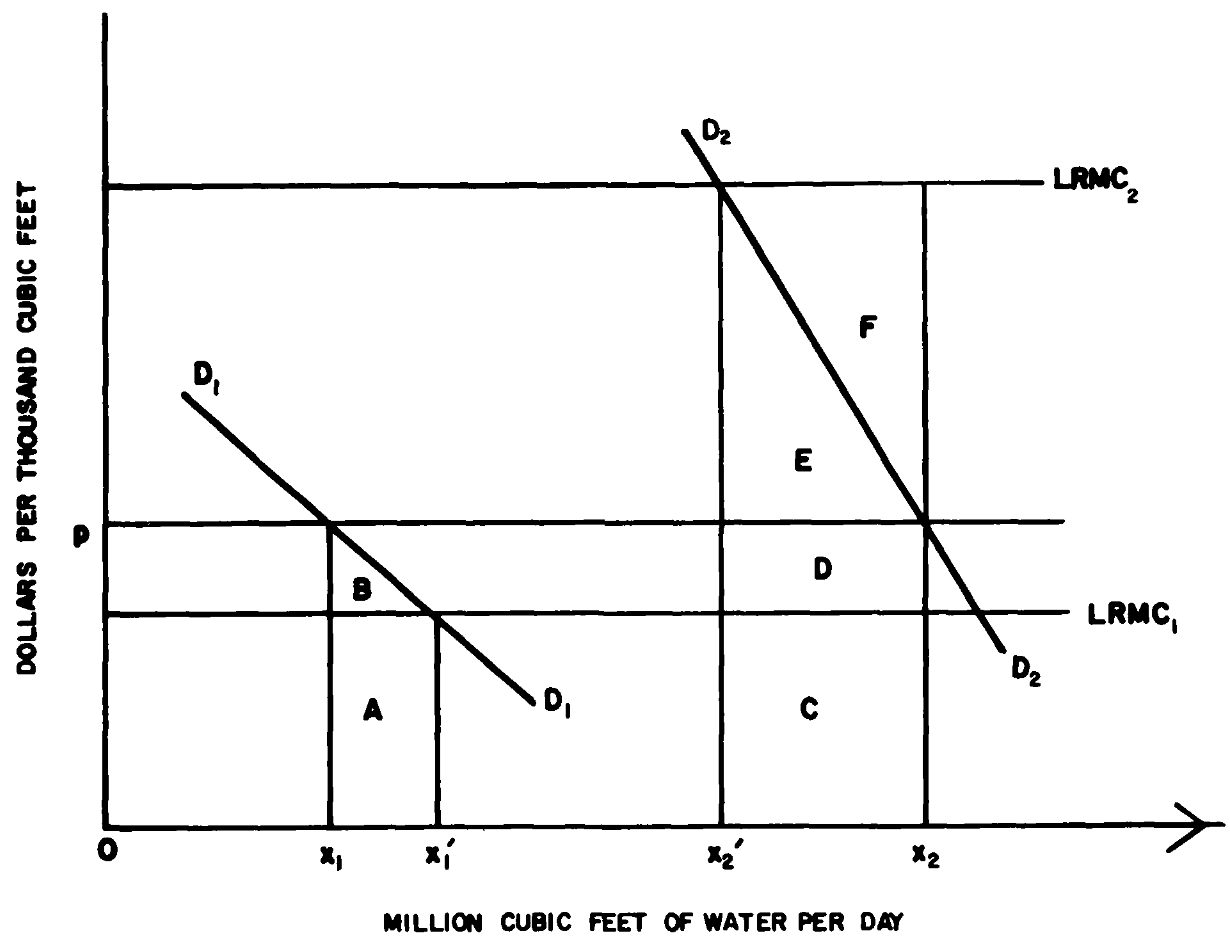

Figure 12. Spatial price differentiation.

to get water. To clarify the impact of this characteristic, consider two customers located near each other along the same distribution main. In the short run, if the main is being used to transport water to these two customers and Customer 1's rate of consumption increases, then it costs more per gallon to deliver water to both Customers 1 and 2. This follows from the Hazen-Williams formula, Eq. 1. As a result, Customer 2 experiences an external diseconomy. Economic principles dictate that Customer 1 should bear the full marginal costs of an increase in his consumption, including those forced upon Customer 2. In this situation, where marginal costs differ significantly from average costs, charging a uniform price, or even apportioning the total cost of a given water supply system component shared by certain customers among those customers on the basis of their consumption, will result in economic inefficiency.

In the long run, an increase in consumption by Customer 1 might require expansion of the distribution network to serve him. Again, economic principles dictate that he should bear the full costs of the expansion. However, there are three factors that complicate the matter:

1. Water transmission and distribution mains are characterized by economies of scale in their capital costs with respect to capacity, so the expansion would probably be built with excess capacity. Only some arbitrary portion of the expansion cost could be properly allocated to Customer 1.
2. Even if the distribution system serving Customers 1 and 2 is expanded solely to meet the growth in demand by Customer 1, Customer 2 receives the benefit of the potential to take water at an increased rate because of the excess capacity created, and should be charged for this.
3. The larger distribution system capacity will allow water to be transported at a lower per unit energy cost. Hence, Customer 2 will reap some of the benefits of the larger main, an external economy.

Spatial Cost Model

All of these factors can be properly tied together through the development of a spatial cost model. It is based on the conceptual representation of the water supply system shown in Figure 4.

The model has a planning horizon of T consecutive time periods. The objective is to maximize the present value (i.e., at time t=0) of total benefits less total costs in periods 1 through T. The decision variables are the prices to be charged in each pumping district j in period t, denoted p_{jt} (or equivalently, the quantities demanded in each pumping district j in period t, denoted y_{jt}) and the size of the increment to capacity on each link (i,j) to be constructed at the beginning of period t, denoted D_{ijt}.

The total benefits at each demand node j in period t may be obtained by integrating the inverse demand function from 0 up to y_{jt}, the quantity demanded at node j in period t:

$$(12) \qquad B_{jt} = \int_0^{y_{jt}} p_{jt}(y)dy \qquad j = 1, \ldots, N; \; t = 1, \ldots, T$$

where y is a dummy variable of integration with units of quantity demanded per time period (e.g., million gallons per day). The present value of the stream of total benefits may be written as

$$(13) \qquad TB = \sum_{t=1}^{T} \sigma_t \sum_{j=1}^{N} B_{jt}$$

where σ_t is a discount operator of the form $\sigma_t=(1+r)^{-t}$, where r is the social rate of discount.

The costs of providing water to customers in this model include the capacity, operating and maintenance costs of treating the water, and the capacity, operating and maintenance costs of transporting it to the various nodes. Assume one central treatment plant at the source (designated node 0) at which the cost of treating the water is a function of the quantity of water produced:

$$(14) \qquad b_{0t} = b_{0t}\left(\sum_{j=1}^{N} x_{0jt}\right)$$

where x_{0j} is the flow of water in the link from node 0 to node j.

The cost of passing water through any link in the supply system between two nodes i and j in period t is a function of the rate of flow and the effective diameter of the link at that point in time, denoted ED_{ijt}, as well as the height to which the water must be raised, the cost of power, and the pumping efficiency. For simplicity, the cost relationship is abbreviated as

$$(15) \qquad c_{ijt} = c_{ijt}(x_{ijt}, ED_{ijt}) \text{ for all } (i,j), t$$

Suppose that the size of the increment in capacity added to link ij in period t is D_{ijt}. Depending on the nature of the facilities being augmented (e.g., adding a parallel pipe, expanding a pumping station, etc.) the cost of the capacity expansion C_{ijt} may also depend on the facilities that are already in place:

$$(16) \qquad C_{ijt} = C_{ijt}(D_{ijt}, ED_{ijt})$$

All costs incurred in period t must be discounted to present value terms. Hence, the objective function may be written as

$$(17) \qquad \text{maximize} \quad \sum_{t=1}^{T} \sigma_t \left[\sum_{n=1}^{N} \int_0^{y_{jt}} p_{jt}(y) - \sum_i \sum_j c_{ijt}(x_{ijt}, ED_{ijt}) - b \sum_i x_{0it} - \sum_i \sum_j C_{ijt}(D_{ijt}, ED_{ijt}) \right]$$

Assume that there is no significant change in storage in the treated water supply system from one period to the next. At any node for any period, water can either be demanded or passed on through transmission links to "downstream" nodes. That is, the water flowing into any node must equal the water flowing out plus the water consumed at the node:

(18) $$\sum_{i=0}^{N} x_{ijt} = \sum_{i=0}^{N} x_{jit} + y_{jt} \qquad j=1,\ldots,N; \; t=1,\ldots,T$$

Continuity is required for capacity additions; that is, the capacity (expressed as an effective diameter) available in link (i,j) at the beginning of period t+1 must equal the capacity available at the beginning of period t plus whatever augmentation is provided at the beginning of period t+1. This relationship was given in Eq. 7 and for notational simplicity may be expressed as follows:

(19) $$ED_{ijt+1} = ED_{ijt+1}\,(ED_{ijt}, D_{ijt+1}) \text{ for all } (i,j), \quad t = 0\ .\ .\ .\ ,\ T-1$$

The following additional simplifying assumptions are made: Transmission capacity expansion may only take place on existing links, and all flows and capacities are non-negative. Depreciation is ignored. Expansion of treatment plant capacity is also ignored. (It could easily be incorporated by representing treatment as a link from some artificial node to the source node with the cost of passing water "through" that link being the treatment cost function.)

*Eqs. 17, 18 and 19 comprise the spatial cost model. Eqs. 17 and 19 are non-*linear. The model can be solved for a reasonable number of variables using any of several non-linear programming algorithms or by solution of the Kuhn-Tucker conditions derived from the partial derivatives of the Lagrangian written for Eqs. 17, 18 and 19. The Kuhn-Tucker equations yield two principal conditions necessary for optimality:

1. The price to be charged in any pumping district j in period t is the marginal treatment cost plus the marginal cost of transporting the water from the source to that pumping district.
2. The present worth of an addition to capacity that becomes available at the beginning of period t should equal the present worth of the terminal or salvage value at the end of the planning horizon plus the contribution to the objective function of the changes in both the operating and capital cost streams from that period through to the end of the planning horizon.

Hence, in applying the model and its conclusions, the interactions between capital and operating costs given by Eqs. 2 and 7 must be known for the major components of the water supply system.

Fitting Observed Data-An Example

Unfortunately, for various reasons, water supply system components do not behave exactly according to the dictates of theory. Yet, if the observed phenomena can be approximated by Eqs. 2 and 7, they can provide guidelines for planning purposes. To understand how this process may be carried out, consider the following example. Figure

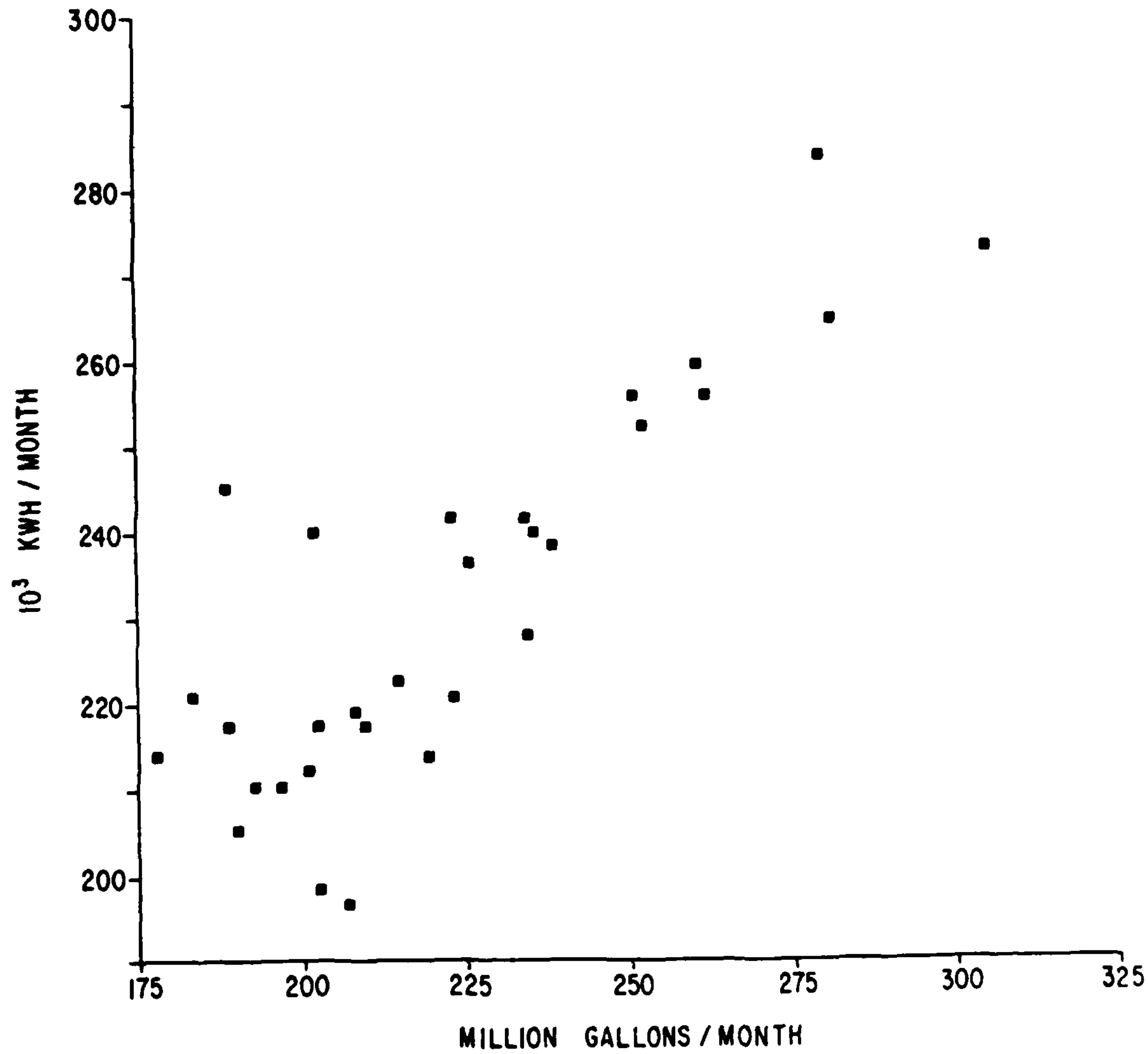

Figure 13. Electric power consumption versus pumpage for a booster pumping station.

13 shows the pumping and electrical consumption data for a booster pumping station. Various types of curves were fitted to the data, and the equation of best fit was

(20) $$c(x) = 5553.37 + 37.5798x^2$$

where x has units of million gallons per day and c has units of kwh/day. In principle c(x) should approximate the sum of static lift energy requirements (a function of x), dynamic head energy requirements (a function of $x^{2.85}$), and a constant energy requirement for overhead. Overhead may include such things as heat and lighting for the pumping station and the power required to maintain an emergency backup generator on "hot standby." If the curve fits, it should approximate the sum of these functions and there should be corresponding slopes for the mean value of x:

(21) $$c(\bar{x}) = b_1\bar{x} + b_2\bar{x}^{2.85} + C$$

(22) $$c'(\bar{x}) = b_1 + b_2\ (2.85)\ \bar{x}^{1.85}$$

where C is the overhead power requirement and b_1 and b_2 are coefficients. In practice, C will absorb unknown factors. Equs. 21 and 22 contain three unknowns that must be solved for: b_1, b_2 and C. However, the coefficient for static lift energy requirements b_1 may be derived from observations of h_s, the difference in elevation between the pumping intake and the discharge at the end of the link, and assumptions about pump efficiency:

(23) $$b_1 = k_e\ h_s\ f^{-1}$$

where k_e is a coefficient (in this case equal to 8.34×10^6 lbs/2655223.28 ft.-lbs./kwh) and f is the pump efficiency. For the data shown in Figure 13 h_s=75 feet and f is assumed to be 0.8, hence

(24) $$b_1 = (75 \text{ft.})\ k_e\ (0.8)^{-1} = 294.47$$

For $\bar{x}$=7.56 mgd, $c(\bar{x})$=7702 kwh/day from Eq.20. From Eqs. 21 and 22, b_2=2.2764 and C=4749 kwh/day. From the Hazen-Williams equation,

(25) $$b_2 = 2.2764 = k_e\ h_f\ f^{-1}, \text{ where}$$

(26) $$h_f = (10.6)\ C^{-1.85}\ x^{1.85}\ ED^{-4.87}$$

Assuming f equals 0.8 and C=100, this yields

(27) $ED^{-4.87}\ l = 274.137$

for $\bar{x}$=7.56 mgd. If l equals 2,000 feet, then ED equals 1.504 feet, or 18 inches.

This approach enables the water supply engineer or planner to develop the parameters necessary to look at the relationship between current capacity, operating costs and capacity expansion. The coefficients can be checked against different parts of the empirical equation fitting the data. For example, for x=6.5 mgd, b_2=2.305, and ED=1.5000 feet. If there is sufficient data on flow rates and hydraulic gradients at the 2 points in question (at the pump intake and at the discharge at the end of the link), the relationship can be established from this.

These formulations are not a substitute for experience nor should they be approached blindly, but they do provide a foundation for analysis so that the spatial aspects of water supply costs can be given the proper consideration in design and pricing of water supply systems.

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VERIFICATION OF MAPS COST ESTIMATING PROCEDURES

by

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Background

The MAPS (Methodology for Areawide Planning Studies) computer program is a multipurpose program developed at the U. S. Army Engineer Waterways Experiment Station (WES) for use in Corps planning level water resources studies. The program is most commonly used to make cost estimates for comparisons of many typical facilities (referred to as "modules" in MAPS) such as dams, force mains, pump stations, open channels, storage tanks, tunnels, water treatment plants, and wellfields. Additional capabilities include preliminary design, simulation, and economic analysis.

The cost functions contained in the MAPS design modules have been synthesized using the most up-to-date cost data available. Virtually every cost function has been cross-checked against data collected from several sources. Each time the program has been used in a study, the program developers at WES have encouraged the users to check the MAPS estimates against actual costs of facilities in the study area to ensure that the calculated cost estimates are appropriate for the study. Therefore, the program has been independently checked by several Corps of Engineers Districts and their consultants.

Nevertheless, a systematic study had never been conducted to verify the MAPS cost estimates against cost data not used in the initial development of the program. This type of verification is usually required for acceptance of computer programs regardless of their intended use.

Purpose

The purpose of this study was to verify the MAPS cost estimating procedure against an independently determined set of cost data. From this kind of analysis it is possible to: (a) determine the accuracy of the individual modules, (b) identify and correct minor shortcomings of the program, and (c) identify potential major program modifications and additions.

This paper has been prepared to present the results of the verification study, thereby providing MAPS additional credibility with both planners and estimators. In addition, readers should gain a better appreciation of the problems associated with planning level cost estimations and a better understanding of the accuracy of the resulting cost estimates.

The reader is referred to Walski (1980a) for an overview of MAPS, Walski (1980b) for a description of the estimating philosophy used in MAPS, and the Corps Engineer Manual 1110-2-502 (U. S. Army, 1980) for the documentation and user's guide.

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Approach

The approach used to conduct the study can be conveniently divided into five steps:

1. Collect design and cost data for individual projects.

2. Make cost estimates and compare with actual cost data.

3. Adjust design data to correct problems with initial estimate and reestimate the cost.

4. Modify procedures with independent data.

5. Make final comparisons.

Each of these steps is described in more detail in the following paragraphs and are then used as a basis for development of general guidelines for conducting verification studies.

It was determined that the best source of data would be an engineering firm with considerable experience in designing a wide variey of water resources projects. Subsequently data were purchased from the firm of CH2M-Hill, which will be referred to for the remainder of the paper as "the contractor." The point of contact with the contractor was the firm's Gainesville, FL office, but data were supplied from projects throughout the country. The contractor provided two distinct types of data: (a) design parameters required as input for MAPS, and (b) actual costs of projects for verification purposes. In most cases, data were provided for five projects for each type of facility. The exceptions to this were open channels and tunnels where the contractor did not have sufficient data, and pump stations where data for one of the facilities were discarded due to inconsistencies.

Initially, the data were entered into the MAPS program. Where the data were not complete, MAPS default values were used. Actual unit prices for individual items were entered whenever they were available, although in many cases the MAPS estimates of unit prices were used. The program was then run for each facility.

Cost estimates for these initial runs were compared with cost data provided by the contractor. Originally all costs were expressed in year-of-construction dollars. Where bids for a given project were available, the low bid was used as the "actual" cost. In a few cases in which the low bid was significantly lower than the engineer's estimate and the other bids, the engineer's estimate, based on detailed plans and specifications, was used. Where bid tabulations were not available, the engineer's estimate, as opposed to the bid price, was used. Comparisons were made solely on the basis of construction costs. Initially, it was hoped that adequate data would be available to verify MAPS operations and maintenance (O&M) cost estimates. However, because of the manner in which utilities generally keep O&M cost records, it was not possible to determine or compare O&M cost for individual facilities or components.

In most cases, the initial MAPS cost estimates were not sufficiently close to the actual costs to be acceptable. There were two reasons for this. The first involved inadequacies in the data and/or special design problems. For example, the actual construction costs of the water treatment plants included the costs of intake structures, which are not considered as part of the design by MAPS. Also, in one particular case, no note was made of the fact that special drilling equipment was required for one of the wells. In these instances, the input to MAPS was adjusted to account for the special condition or the costs of special facilities (e.g., intakes) were estimated by other means and then added to the MAPS estimate. In some cases, the costs were not adjusted because the MAPS user (in a typical planning study) would not have access to the specific data needed to adjust the costs; hence, the adjustment would not result in a correct reflection of the accuracy of MAPS.

The second source of error in the initial estimates existed in MAPS itself. This was attributed to three general causes: (a) programming errors, (b) limited range cost functions, or (c) unsuitable cost functions. The programming errors found were corrected. In some instances, cost functions were found to be appropriate only for a limited range of sizes or types of facilities. For example, the cost functions for siphons in canals were found to be good only for large siphons, so additional data were used to extend the range to flows as low as 1 cfs (0.0283 m^3/s). In another instance, the wellfield piping cost algorithm, which was only appropriate for wells arranged in a circle, was modified to account also for wells arranged in a line. Finally, where a cost function was found to be weak, it was replaced using a separate data set to develop the new function. With only a few exceptions (where data were very scarce) cost data used for the verification study were not used in modifying the cost functions.

Once all adjustments were complete, a final run of each MAPS program module was made. The results of these runs are presented and discussed in the body of this paper. In a few cases, the error in the final runs was still fairly high. At this point, the MAPS estimate and the actual cost were compared with other data for that type of facility. This was done to determine if the facility was an unusual facility so that the cost could be considered as an "outlier." In all of these outliers, the actual cost was found to be significantly higher than the MAPS estimate or other sources of data. It is believed that these high costs resulted from including other items, which are not usually considered as part of the facility, in the project cost.

Accuracy

In evaluating the MAPS cost estimating procedure, the key question that must be asked is, "How accurate should cost estimates for planning studies be?" There is no simple answer to this in Corps of Engineer regulations or manuals. For government estimates based on detailed plans and specifications, the regulation on engineering contracts (ER 1180-1-1, U. S. Army, 1969) requires that for Civil Works projects all bids be rejected if the low bid is more than 25 percent higher than the government estimate. Certainly, a planning level tool such as MAPS should not be required to be more accurate than a government estimate based on detailed plans and specifications. As a point of reference, it is not uncommon for bids on a given project to vary by as much as 50 percent between the high and low bidder.

The Corps' cost estimating manual (EM 110-2-1301) (U. S. Army, 1980) does not suggest an expected accuracy for planning level estimates. It does state that for small (<$10 million) projects in the survey and review stage, 25 percent should normally be allowed for contingencies. EM 110-2-1301 further states, "The degree of accuracy and precision in estimates at various stages of design will be considered in light of the use thereof, such as comparison and elimination of alternatives, weeding out of less practicable solutions, etc." It is important to remember that the principal use of estimates in planning studies is for comparison of alternatives. Since the estimating procedures in MAPS are internally consistent, the program will generally be serving its purpose, in that relative costs of similar alternatives will be accurate, even if there are some inaccuracies in absolute costs. Nevertheless, absolute costs are used as the basis for the estimates here since comparisons in planning studies are often made between different types of facilities.

In light of the above discussion, the MAPS estimates should be considered sufficiently accurate as long as they are within 25 percent of the actual costs and corrections are made to the estimates for any extraordinary conditions not directly accounted for by MAPS.

Overview

The following sections contain descriptions of the verification results for each type of facility. The MAPS cost estimates are compared first with the contractor data then with other literature data. After this, the process of verification for cost functions in general is discussed. The description of the projects presented in the following sections is limited. For additional details, the reader is referred to the MAPS verification study final report (Lindsey and Walski, 1982).

Dams

The five dam projects for which the contractor provided data consisted of four earth dams (one with a concrete spillway section and one with a separate spillway) and a concrete diversion dam. The dam heights ranged from 17 ft (5.18 m) to 217 ft (66.1 m), and spillway capacities varied from 1800 (51 m^3/s) to 35,000 cfs (990 m^3/s). Two of the dams had gated spillways. None had hydroelectric generating facilities. Four of the MAPS estimates were within 14 percent of the contractor's while one differed by 40 percent.

As a result of this portion of the study two shortcomings in the program were identified. First, the cost function for spillways does not account for possibly significant variations in the costs of different types of spillways (e.g., spillway in dam, spillway separate from dam, drop inlet). Unfortunately, the program developers could not locate a set of spillway cost data for which the type of spillway is identified. Second, deficiencies in the manner in which riprap volumes were calculated were detected and modified.

The MAPS estimating procedure was shown to be a better predictor of costs than functions based on storage volume of the reservoir behind the dam. Additional verification of the dam estimating procedure was reported by Walski and Pelliccia (1981).

Force Mains

The five force main projects for which the contractor provided data consisted of two ductile iron and three prestressed concrete cylinder pipelines. Diameters ranged from 30 in. (750 mm) to 84 in. (2100 mm) and lengths varied from 2,200 ft (670 m) to 74,500 ft (22,707 m). The MAPS estimates were within 20 percent of actual costs for three mains. On another project, for which there were no relocations and bidding was described as "highly competitive," the MAPS estimate was high by 32 percent. The MAPS estimate was low by 41 percent for a short line located in a highly congested industrial area. This project required extensive dewatering and had a very high mobilization cost per unit pipeline length.

In general, MAPS estimates compared favorably with other published cost data. However, to improve future accuracy a new function relating trench bottom width to pipe diameter was developed and the function relating unit cost of excavation to depth of excavation was modified. Also, the regression equation relating the cost of several types of concrete pipe to diameter and pressure rating were rederived from raw data provided by the U. S. Bureau of Reclamation.

Pump Stations

Data were provided by the contractor for four stations having a range of capacities of 1.1 mgd (0.482 m^3/s) to 160 mgd (7.01 m^3/s). MAPS estimates for two of the pump stations were accurate to 4.5 and 14.3 percent, while those for two others were low by 48 percent. However, these two latter cases represented somewhat unusual pump stations. The first was a completely subterranean structure and included a chemical injection unit and a remote monitoring system. The second was an elaborate structure described as being "architecturally matched to a nearby church." Unfortunately data were not available for separating the costs of these stations into individual items so it was not possible to precisely identify the source of the error.

Since there was a significant discrepancy in two of the estimates, additional verification of MAPS costs with data from other sources was conducted. In general the agreement between MAPS and the other sources was quite good and the two stations for which the MAPS estimates were excessively low proved to be outliers.

Changes made to the MAPS pump station estimating procedure as a result of the verification study included: (1) inclusion of an explicit factor to account for number of pumps at a pumping station, (2) improved wet well and small structure cost functions, (3) inclusion of a correction factor for wastewater pumping stations and (4) increased costs for piping, valves, and manifolds.

Open Channels

The contractor provided data only for a single enlargement project and a number of canal structures. Cost comparisons were, therefore, made only for excavation (length 2.9 mile (4645 m), bottom width 51 ft (15.5 m)), siphons (flow range 1.9 cfs (0.54 m^3/s) to 239 cfs (6.7 m^3/s)), radial gates (area range 42 ft^2 (3.9 m^2) to 171 ft^2 (15.8 m^2)) and drop structures (height of

drop range 3.7 ft (1.1 m) to 15.1 ft (4.6 m)). The MAPS estimates were generally within 10 percent of actual costs.

The range of sizes for which MAPS estimates are accurate for drop structures, siphons, and radial gates was expanded to include smaller siphons and radial gates and larger flows (up to 1000 cfs (28 m^3/s)) for drop structures.

The accuracy of earthworks calculations were found to be highly dependent on the accuracy of the elevations input to the program, especially for small canals. Elevations should be specified to the nearest foot when possible.

Storage Tanks

The contractor provided data for two concrete ground level tanks (5 MG* (18,925 m^3) and 8 MG (30,280 m^3)), two elevated steel tanks (0.1 MG (378 m^3) and 1.0 MG (3785 m^3)) and one steel standpipe (0.75 MG (2839 m^3)). The MAPS estimates were within 20 percent of the actual cost for three of the tanks. For the smaller concrete tank, the MAPS estimate was high by 45 percent. This appeared to be due to the fact that the tank was part of a much larger project on which the bidding was described by the project engineer as "highly competitive." Therefore much of the mobilization cost was probably absorbed into other items. In the case of the large elevated steel tank, the MAPS estimate was low by 28 percent. This was the result of a relatively sophisticated tank design.

Estimates prepared using MAPS agreed very well with data from other sources indicating that the two projects for which the MAPS estimates were not acceptably accurate were outliers. During this study new cost functions for consideration of buried concrete tanks were added to MAPS.

Tunnels

The contractor provided data for two 7500-ft (2286 m) machine bored tunnels having diameters of 5 ft (1.5 m) and 6 ft (1.8 m). Even though the cost functions in MAPS were derived for tunnels with widths greater than 10 ft (3.05 m), the MAPS cost estimates were within 3.5 and 17.2 percent of the actual costs. In the case of the tunnel with the larger error, the rock quality designation (RQD) was 15. The MAPS functions were derived for RQD values between 40 and 100.

The only modifications made to the program as a result of the study were to the limits on RQD and unconfined compressive strength at which the program shifts from one set of cost functions to another.

Water Treatment Plants

The contractor provided data for three conventional surface water plants and two softening plants having a range of flows from 6 mgd (0.26 m^3/s) to 125 mgd (5.5 m^3/s). There were some problems in making the comparisons in that the version of MAPS existing at that time did not include costs for intake structures, clearwells, administration and laboratory buildings, or

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* MG = million gallons

sludge handling facilities. Therefore, these costs were "hand calculated" using data from Gumerman, Culp, and Hansen (1981) and added to the MAPS estimates.

Three of the MAPS estimates were within 12.3 percent of the actual costs. One MAPS estimate was low by 18.3 percent. This estimate resulted because the plant design included two very large buried concrete tanks constructed in an area where the water table was near the surface. For another plant, the MAPS estimate was low by 21.6 percent. In this case the plant was actually only the first stage of a much larger (by a factor of 3) plant, and the yard piping and some other facilities were sized for the ultimate flow.

MAPS estimates were prepared for conventional plants with capacity between 1 mgd (0.0438 m^3/s) and 200 mgd (8.76 m^3/s). In general the MAPS estimates were low by approximately 20 percent, reflecting the fact that intakes, administrative and laboratory buildings and sludge handling facilities were not included in the MAPS estimate. These facilities (with the exception of intake structures which are being considered separately in MAPS) are now included in the program. A recent comparison of the upgraded MAPS program with these additional sources of cost data shows good agreement.

Wellfields

The contractor provided data for five wellfields having capacities ranging from 2.2 mgd (0.10 m^3/s) to 72 mgd (3.15 m^3/s). One MAPS estimate was within 7 percent of the actual cost. Two of the MAPS estimates were lower than the actual cost by 23 percent. The first occurred because one of the wells was drilled and then abandoned, and an unusual casing configuration was used. The second was due to oversized piping in the wellfield (probably due to installing capacity for later expansions) and expensive housing for the wells (>$100,000 per well) since each well was pumping 15 mgd (0.66 m^3/s). The MAPS housing costs are not a function of capacity. The MAPS estimate and actual cost for another wellfield differed by 32.9 percent. This was due to a combination of a considerable length of 72 in. (1800 mm) pipeline, which was significantly oversized for the wellfield, plus some extremely elaborate control equipment. The final wellfield yielded the poorest agreement between predicted and actual costs in the entire study with an error of 61.7 percent. This was a very unusual wellfield in that (1) it was drilled through a highly productive polluted aquifer, thus requiring extensive casing and concrete, (2) bids were accepted from only three contractors, and (3) special corrosion resistant pumps were specified along with considerable extra equipment.

Even though MAPS estimates compared well to those found in other references, several modifications were made. As a result of this study, it is now possible to specify "tubular" or "gravel packed" wells in unconsolidated material and well housing can be described by the user as "simple" or "elaborate." It is also possible to align the wells in a row instead of only in a circle as was possible with earlier versions of MAPS. Since control equipment at wells can represent a significant fraction of the cost, it is now possible for the user to specify "simple," "elaborate" or "no" controls, or enter the cost of controls directly to the program.

Results of Verification

Comparisons were made between actual costs and MAPS estimates for 35 different facilities. A summary of the results of these comparisons is shown in Figure 1. All of the points would fall on the 45-deg line in Figure 1 if the program were perfectly accurate. The points are all fairly close to the line, indicating a high level of correlation between the actual and MAPS costs.

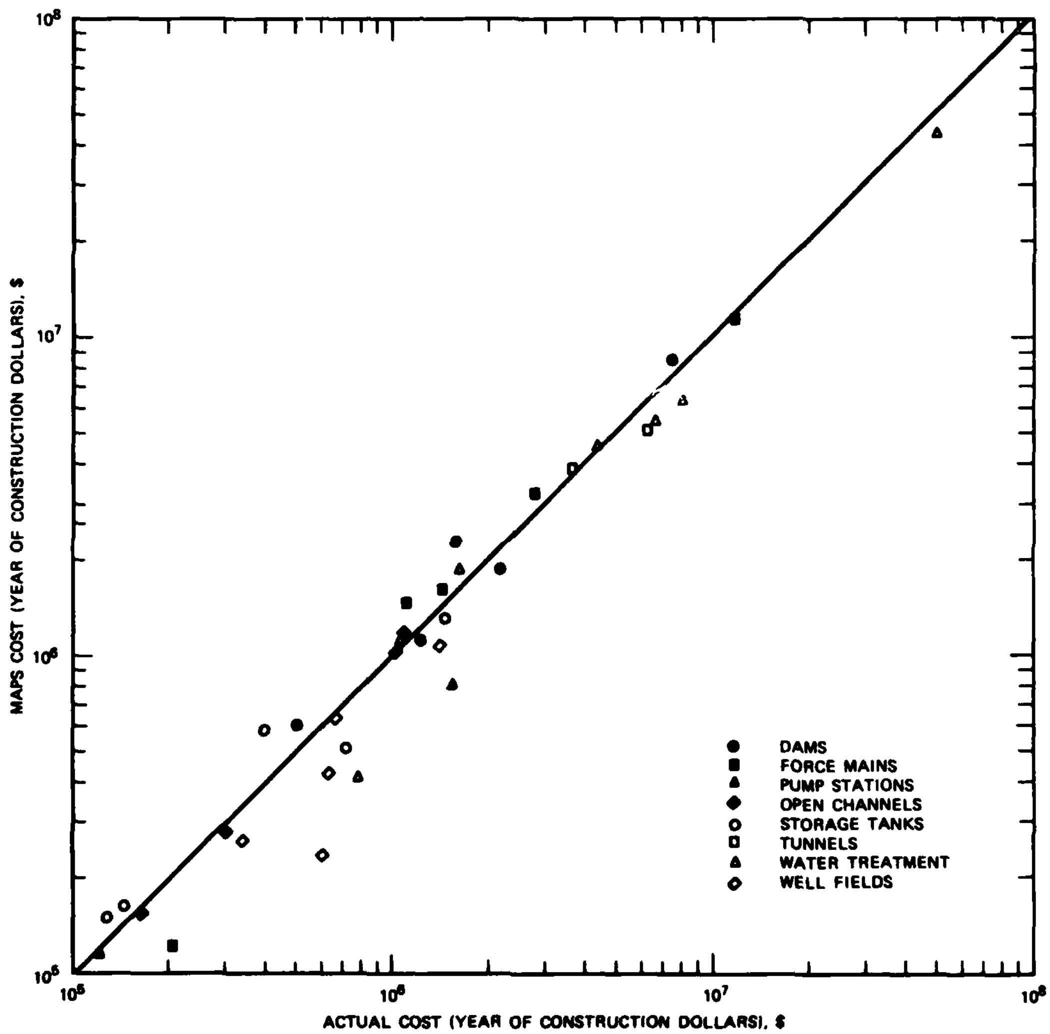

Figure 1. Summary of actual and MAPS costs

The geometric mean percent error for the 35 facilities was 13.9 percent. Existing guidance in Corps of Engineers regulations and manuals on cost estimating indicates that estimates are considered accurate if they fall within 25 percent of actual costs. The MAPS cost estimates for 75 percent of the facilities fall within this 25-percent range. The cost estimate for only one facility differs by more than 50 percent. The correlation coefficient between actual and MAPS costs was 0.967.

Comparison of the MAPS cost functions with other cost functions showed a high degree of consistency. This indicates that the facilities for which MAPS produced poor estimates were unusual cases which MAPS should not be expected to handle. Such facilities could conceivably be identified beforehand and considered separately. Nevertheless, MAPS is considerably better than generalized cost functions in accounting for the many variables affecting costs, as generalized cost functions usually have only one or two independent variables. In general, the MAPS computer program produced cost estimates of acceptable accuracy for planning studies.

As the result of this study, many of the shortcomings of the program (e.g. limited range of cost functions and difficulty in accounting for some important variables) were identified and corrected. In the case of water treatment plants, the program has been upgraded independently since this study.

Guidelines for Cost Verification Studies

The guidelines developed in conducting the MAPS Verification Study, should be applicable to other studies involving verification of cost estimating procedures. A recommended step-by-step procedure for conducting a verification study is presented below.

1. Gather data on a set of facilities which are significantly different from each other. The data should contain a description at least in sufficient detail to use the costing procedure but preferably in greater detail. The cost data should be based on actual construction cost, if that is not available, actual bids, and if that is not available, detailed engineering estimates. Costs should be disaggregated to account for sub-items to the extent possible, otherwise one can only verify total facility costs. It is important that the facilities represent a variety of different construction types. For example, if a procedure for determining the costs of wells is verified using data only for 8 in. (200 mm) diameter wells drilled to 100 ft (30 m) in unconsolidated material, it is not possible to infer anything about the accuracy of the procedure for 12 in. (300 mm) diameter wells drilled to 500 ft (152 m) in rock. Verification study results are only as good as the initial data.

2. Given the description of the facility and information on price levels, estimate the costs and compare with the actual costs. The most important comparison is between total predicted and total actual costs, but the costs of individual items should also be compared. Usually an error in the total cost can be traced to error in an individual item. If the error is spread uniformly through the items, it is most likely caused by incorrect price levels resulting from very competitive or very non-competitive bidding. If the totals agree but individual items differ, there are two possibilities. The first is that the accuracy of the final answer is a coincidence due to compensating

errors. In this case the source of error in the individual items should be identified. The second is that the error is due to different bookkeeping procedures between the actual and predicted cost. For example, the manifold and valves at a pump station may be considered as mechanical equipment in the bid, but as miscellaneous equipment in the estimating procedure being tested. Only by examining the project data in detail will the source of error be identified.

3. Make adjustments to input data and repeat the estimating procedure. (Note that the adjustments are made to the data not the procedure.) For example, one might change a filter loading rate from 5 gpm/ft^2 (0.0034 m/s) to 2 gpm/ft^2 (0.0013 m/s) or specify a cost function for an "elaborate" structure rather than a "simple" structure. If the purpose of the study is to "validate" the model, then the study is complete at this point. Since, in most "verification" studies the purpose is to improve the estimating procedure, the sources of errors should be identified and corrected as described below.

4. The cost estimating procedure should be modified using data other than that described in step one. It is easy at this stage to "fudge" the cost functions to agree with the actual costs. If this is done, the study is reduced to "calibration" rather than "verification." The results of step three should be used to identify portions of the estimating procedure requiring improvements, but these improvements should be made using a separate data source. An example of the type of change made in this portion of the study would be development of a procedure to estimate costs for piping between wells for a non-circular well configuration.

5. Once modifications have been made, the actual and predicted costs should be compared and the results documented.

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ESTIMATION OF COSTS FOR SMALL WATER SYSTEMS

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COST ESTIMATION PROBLEMS

The accuracy of cost estimates for water treatment processes is naturally dependent upon the appropriateness of the information used to develop them. Securing accurate and detailed cost and operational data for the variety of water treatment processes available to small water systems is paramount to the satisfactory development of cost estimates for both capital expenditures and operations and maintenance requirements. Unfortunately, the information available on small water systems is very limited since their size and relative lack of support services do not provide the luxury of establishing and maintaining substantial recordkeeping systems. This lack of important basic information complicates the development of planning estimates.

Part of the difficulty in obtaining data concerning small water systems is that the operations are conducted on a scale that does not afford the opportunity to record specific costs for unit processes, especially for operations and maintenance. Operators will frequently keep records relating to operating parameters or water quality, but information relating to labor requirements or receipts for materials purchases on individual unit processes may receive little or no attention. If the utility is publically owned, the employees operating the water utility may divide their time between operation of the water system and other public work functions. If unit processes are housed in buildings which are also used for other functions, it may be difficult to separate housing and energy costs specifically related to the water system.

In addition to the general lack of available data, it is also extremely difficult to extrapolate unit process cost data for larger systems to the small water systems. For many processes, there is an entirely different economy of scale in effect for systems below 1.0 MGD than that which exists for

larger systems. Because of these different economies of scale, a different treatment process for a particular water quality may be most cost effective at different system sizes. Small systems also have a much smaller rate base to draw upon than do larger systems. Thus, incremental costs of added treatment processes result in disproportionate increases to customer costs. Thus, special attention must be paid to alternative treatment technologies to the conventional treatment that may be routine for specific water quality problems in larger systems.

The inclusion of small systems in the provisions of the National Interim Primary Drinking Water Regulations has presented those communities who fail to meet some of the requirements with potentially severe economic decisions. With respect to the difficulties of meeting water quality standards, the potential problems faced by individual small systems are highly variable and the proposed solutions are likely to be very site-specific. Smaller systems typically have less latitude in seeking alternative water supplies and generally will make use of groundwtaer in the immediate vicinity of the population to be served. Thus, they are more subject to local water quality problems than larger systems that would have the ability to look further for better quality sources.

Frequently, the water quality problem that may be encountered by a small system would be the presence of a single contaminant in an otherwise satisfactory water supply. Conventional solutions to these problems would generally involve the addition of standard treatment requiring large capital investment and high operating costs which are traditionally only borne by systems with substantial populations. Alternatively, an economically attractive solution might be the acquisition of a commercially-available unit process, or combination of unit processes (designed to function with minimal operator attention) that will specifically address the contaminant in question. The use of such package or prefabricated facilities will generally produce substantial capital and operations cost savings over conventional treatment works. In specific circumstances, the potential for point-of-use treatment devices in some or all of the homes being served may be economically feasible.

In addition to treatment alternatives, both conventional and innovative supply or distribution techniques might be utilized. Regionalization alternatives, including both interconnections and the sharing of technical or support services might be possible in lieu of seeking a new source.

In very small systems, or under temporary or emergency conditions, the distribution or availability of bulk bottled water might be considered. Under special circumstances, the potential for supplying water of two different qualities, one potable and the other subpotable, through a dual water system might be feasible.

WATMAN MODEL

The National Science Foundation[1] had decided to award a research grant to develop a methodology for economically evaluating alternatives, both conventional and new technology, for water systems to solve water supply and treatment problems. Under the initial study, WESTON developed an efficient and practical systems model named WATMAN, (WATer MANagement model) for technical and economical analysis of various alternatives for long-term water supply management. The model has been found to be flexible enough to simulate almost any configuration of water supply systems.

As originally developed, however, the cost functions available in WATMAN were applicable to water system in excess of 20,000 population. In order to expand the model to be appropriate for small systems and to also incorporate alternatives uniquely situated for small communities, small systems data is now available to be utilized to economically compare various alternatives for small and large systems.[2]

DATA COLLECTION

Important sources of information for this data base included several U.S. Environmental Protection Agency studies of small system operations and treatment unit operations costs, survey results and information provided by the Pennsylvania

Department of Environmental Resources, and equipment manufacturer's information and reviews. These were supplemented by WESTON's own cost estimates, by conducting a survey of bottled water distributors with the assistance of the International Bottled Water Association, and direct contacts with operators of small systems.

The accuracy of cost estimates for water treatment processes is naturally dependent upon the appropriateness of the information used to develop them. Unfortunately, the information available on small water systems is very limited. The studies above are considered the best data presently available for determining such costs. The surveys of bottled water distributors and point-of-use device manufacturers were conducted to provide specific cost information in those areas where data from other sources was particularly scarce.

In order to obtain data for use in the WATMAN model for the bottled water distribution alternative, a survey of bottled water companies was conducted with the assistance of the International Bottled Water Association. One quarter of the 89 companies who were mailed a questionnaire on their operations responded. The questionnaire produced cost information on distribution, facilities, volumes produced and delivered, capacities, operations and maintenance, and on the capabilities of the companies to service small water systems. This information was utilized to analyze bottled water costs as they vary with numbers of customers, volume of delivery, delivery route length, and other factors and provided the basis of the cost function utilized in the WATMAN model.

With the assistance of the Water Quality Association, an industry spokesman organization, cost data and manufacturers information were obtained by the Study Team from eleven manufacturers of point-of-use treatment devices.

Information provided by the manufacturers included capital and installation costs, recommended replacement volumes, cost of replacement units, and instructions on operation and maintenance.

COST FUNCTIONS

Cost functions have been developed for twenty-five unit operations or supply and distribution techniques applicable for small systems for utilization by the WATMAN model to predict capital and operations and maintenance costs for the alternative technologies under consideration. These cost functions take the form of a mathematical expression:

$$Y = a + bQ^{c}$$

where Y is the capital cost in dollars, or operations and maintenance cost in dollars per year, Q is flow in MGD (Maximum daily flow for the community for most capital cost functions and average daily flow for point-of-use devices and all operations and maintenance cost functions), and a, b, and c are values developed from the cost information. Figure 1 shows a plot of the cost functions developed for package complete filtration plants.

The development of the cost functions proceeds from a tabulation of cost data over the range of flows considered. Such a tabulation for package plant construction costs is shown in Table 1. The costs for various flows are plotted on log-log paper. The value of constant a is determined through a trial and error graphical procedure and is equal to that value which when subtracted from the plotted cost curve will result in a straight line fit. If the original cost plot was a straight line, the value of a is zero. Once a has been determined, b and c can be readily calculated. The developed cost function is then checked at various flow values to check its accuracy with the original cost data. A listing of the cost functions developed appropriate to small water systems is provided in Table 2.

In addition to the appropriateness of the original cost data, a second major factor in the accuracy of the cost estimates for water treatment processes is the ability of the methodology to include site specific characteristics in its estimates. The WATMAN model incorporates site-specific factors through two techniques. First, it determines the water demand for the community utilizing specific population and consumption data. It has the capability of estimating up to six different demand categories for three different water types. Secondly, although the unit processes cost functions are expressed as a function of flow, the potential exists to override assumed values for coefficients and exponents of

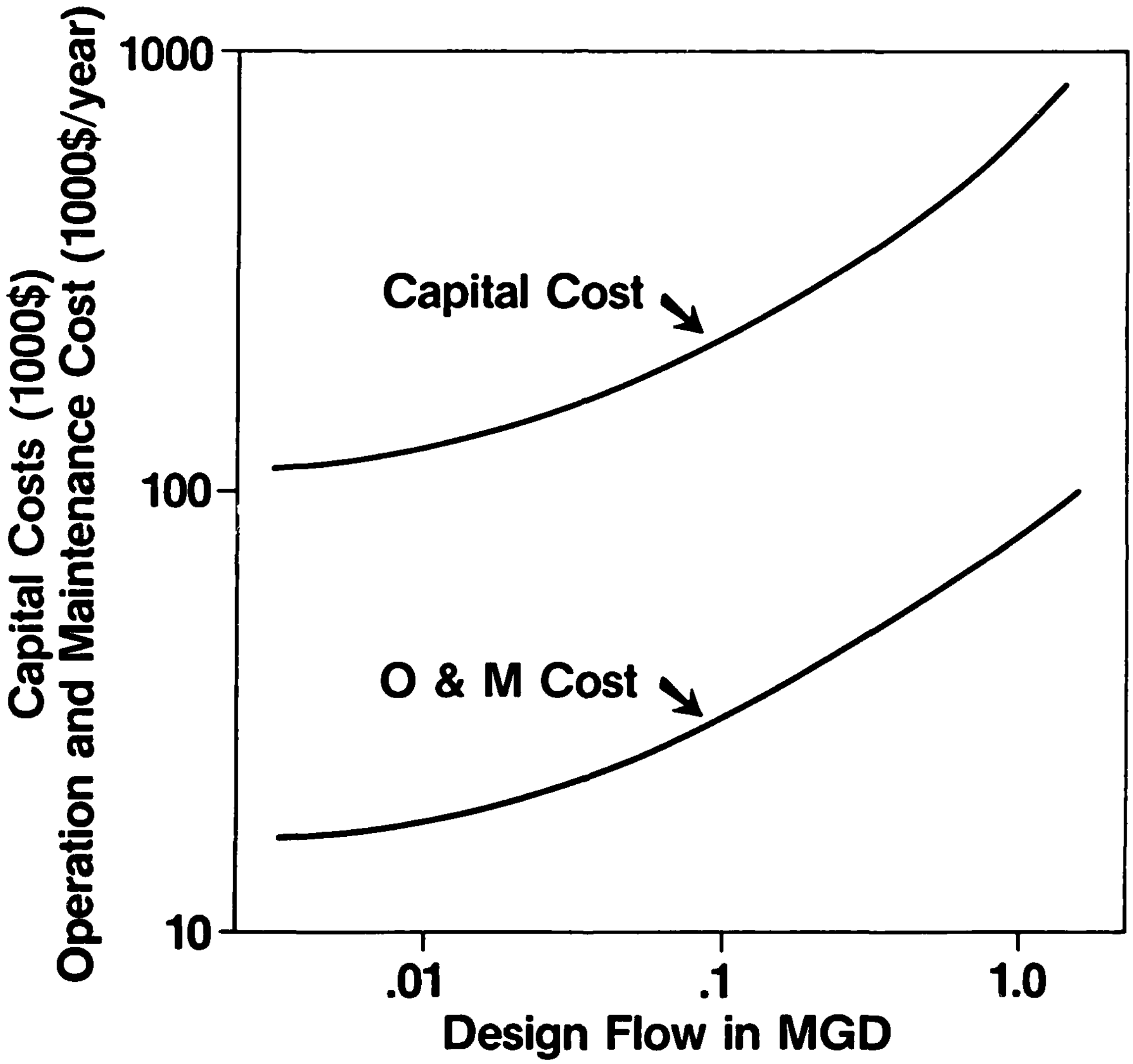

Figure 1. Package filter plant cost functions.

TABLE 1

PACKAGE PLANT CAPITAL COST SUMMARY IN ($)

	PLANT CAPACITY (GPM)						
	10	20	100	200	350	560	700
SITEWORK	400	500	700	1,000	1,400	1,800	2,100
PURCHASE PRICE	24,300	29,400	43,000	97,000	129,000	155,000	162,000
CONCRETE	6,200	7,800	11,600	18,500	25,800	35,100	38,400
LABOR	7,400	8,700	10,100	14,800	16,600	17,900	21,600
PLUMBING/ INSTRUMEN- TATION	25,600	27,600	34,100	34,100	43,600	55,300	76,200
HOUSING	32,900	36,000	44,800	60,200	72,300	72,300	72,300
MISCELLA- NEOUS/CON- TINGENCY	14,500	16,500	21,600	33,800	43,300	50,600	55,900
ENGINEERING/ SUBCON- TRACT	18,900	19,000	24,900	38,900	49,800	58,200	64,300
TOTAL	130,200	145,500	190,800	198,300	381,800	446,200	492,800

TABLE 2

SMALL WATER SYSTEMS COST FUNCTIONS (JUNE 1981)

Unit Process/Methodology	Capital Cost, $	O&M Cost, $/Year)
Impounded Reservoir	$180,000Q_M^{.62}$*	$1160Q^{.54}$*
Raw Water Pumping	$7000 + 3440\ Q_M^{.38}$	$600 + 4900Q^{.91}$
Aeration	$17,000\ Q_M^{.74}$	$8200\ Q^{1.01}$
Chlorination	$12,500 + 51,4700\ Q_M^{.74}$	$4150 + 10,700Q^{.70}$
Rapid Mix & Flocculation	$63,300\ Q_M^{.45}$	$600 + 4600Q^{.59}$
Clarification	$8600 + 180,000Q_M^{.51}$	$2800Q^{.20}$
Gravity Filtration	$670,000Q_M^{.62}$	$2500 + 37500Q^{.70}$
Pressure Filtration	$14,500 + 285,000Q_M^{.56}$	$3200 + 107,000Q^{.77}$
Ultrafiltration	$16,400 + 846,000Q_M^{.92}$	$3200 + 107,000Q^{.77}$
Package Complete Filter Plant	$60,000 + 433,000Q_M^{.43}$	$11,000 + 60,500Q^{.50}$
Granular Activated Carbon	$11,200 + 173,000Q_M^{.67}$	$1460 + 49,200Q^{.94}$
PAC Feed System	$850 + 6600Q_M^{.67}$	$1100 + 28,300Q^{.81}$
Ozone Generation	$61,000Q_M^{.38}$	$9,250Q^{.22}$
Reverse Osmosis	$215,000 + 1,280,000Q_M^{.87}$	$14,400 + 302,000Q^{.95}$

TABLE 2
(continued)

Cation Exchange	$550{,}000Q_M^{.56}$	$114{,}000Q^{.67}$
Anion Exchange	$850{,}000Q_M^{.54}$	$3500 + 520{,}000Q^{1.32}$
Activated Alumina	$675{,}000Q_M^{.60}$	$2700 + 180{,}000Q^{1.34}$
High Service Pumping	$46{,}600Q_M^{.19}$	$45{,}000Q^{.78}$
Water Storage Tanks	$9500 + 2{,}090{,}000Q_M^{1.06}$	$170 + 36{,}800Q^{1.13}$
Bottled Water Distribution		$74{,}550{,}000Q_B$
Point-of-Use (GAC)	$21{,}140{,}000Q_P$	$10{,}460{,}000Q_P$
Point-of-Use (Cation Exchange)	$31{,}820{,}000Q_P$	$10{,}460{,}000Q_P$
Point-of-Use (Anion Exchange)	$62{,}510{,}000Q_P$	$35{,}460{,}000Q_P$
Point-of-Use (Activated Alumina)	$17{,}050{,}000Q_P$	$14{,}140{,}000Q_P$
Point-of-Use (Reverse Osmosis)	$59{,}100{,}000Q_P$	$22{,}120{,}000Q_P$

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*Q_M - Maximum Daily Flow for Community in MGD
Q - Average Daily Flow for Community in MGD
Q_B - Average Daily Distribution of Bottled Water in MGD
Q_P - Average Daily Flow Treated by Point-of-Use Devices in MGD
** - Approximated from a Detailed Bottled Water Distribution Model Analysis

cost functions in order to make the estimates more site specific. For example, if a user has access to cost information he considered more specific than the source data for the cost function under consideration, he could derive his own values of a, b, and c and use them in place of the provided values.

Cost functions have been developed to represent the full costs of a unit process to as great as an extent as is possible. Capital costs represent the complete purchase, construction, and installation costs and include preparatory sitework, purchase price, construction materials and labor, plumbing, pumps, electrical, and instrumentation units, housing, miscellaneous and contingency factors, and engineering and subcontractor fees. For the small systems capital cost functions, the majority have been determined on the design basis of maximum daily flow. Operations and maintenance costs represent the complete annual costs expected to be incurred as a result of the operation of the unit process. These costs include labor, all materials including chemicals, and process and housing-related energy costs.

MODEL DEVELOPMENT

The small system unit process cost functions described above have been incorporated into the WATMAN model. A detailed description of the WATMAN model is provided elsewhere.[(1)(3)] The model is now capable of evaluating the full range of water utility operations for almost any conceivable configuration of alternative processes. With the addition of small water system applications, the following capabilities now exist within the WATMAN model:

1. The system is capable of evaluating both large and small water utilities.

2. The system is capable of handling a conventional water supply management system configuration as well as a multiple water supply management system configuration.

3. The alternative technologies of point-of-use treatment, bottled water distribution and package treatment plants and equipment can be considered for small water systems, in addition to the use of conventional treatment processes, new water sources, and dual water supply previously available in WATMAN.

4. The system is capable of handling a regional water supply management system configuration serving up to 10 communities.

5. The design flow of a unit process may be derived from total average water demand or from interior residential water demand and population data.

SILVERDALE TEST CASE APPLICATION

In order to demonstrate its capability in evaluating alternatives available to small water systems, the WATMAN model was used to conduct economic comparisons of options under consideration in four test case situations. One of these test cases was the W. C. Seidel Water Works which presently serves a populatin of about 150 persons (50 services) in and around the town of Silverdale, located in Bucks County, Pennsylvania. The water system is a private utility, owned and operated on a part-time basis by Mr. and Mrs. W. C. Seidel, who comprise the total staff but are not salaried. Not all residents of the Silverdale area are serviced by the water company, as many have individual wells.

The Seidel Water Works secures its water from four wells, all located within 100 feet of each other. Two pressure storage tanks holding a combined storage of 4,000 gallons are located near the wells. These wells are situated at the lowest elevation of the service area so all water must be pumped from this locatin for distribution. An elevation difference of about 90 feet exists within the system. A map of the system is shown in Figure 2.

The make-up of the service area is essentially 100% residential. The services and wells are not metered so precise figures for demand are not available. The vast majority of pipes in the distribution system have been replaced in the last 15 years so distributin losses can be expected to be minimal. A demand rate of 44 gpcd residential flow and a maximum to average daily flow ratio of 1.8 were assumed to be appropriate for analysis.

Since October of 1979, water samples collected by the Bucks County Department of Health and analyzed by the Pennsylvania

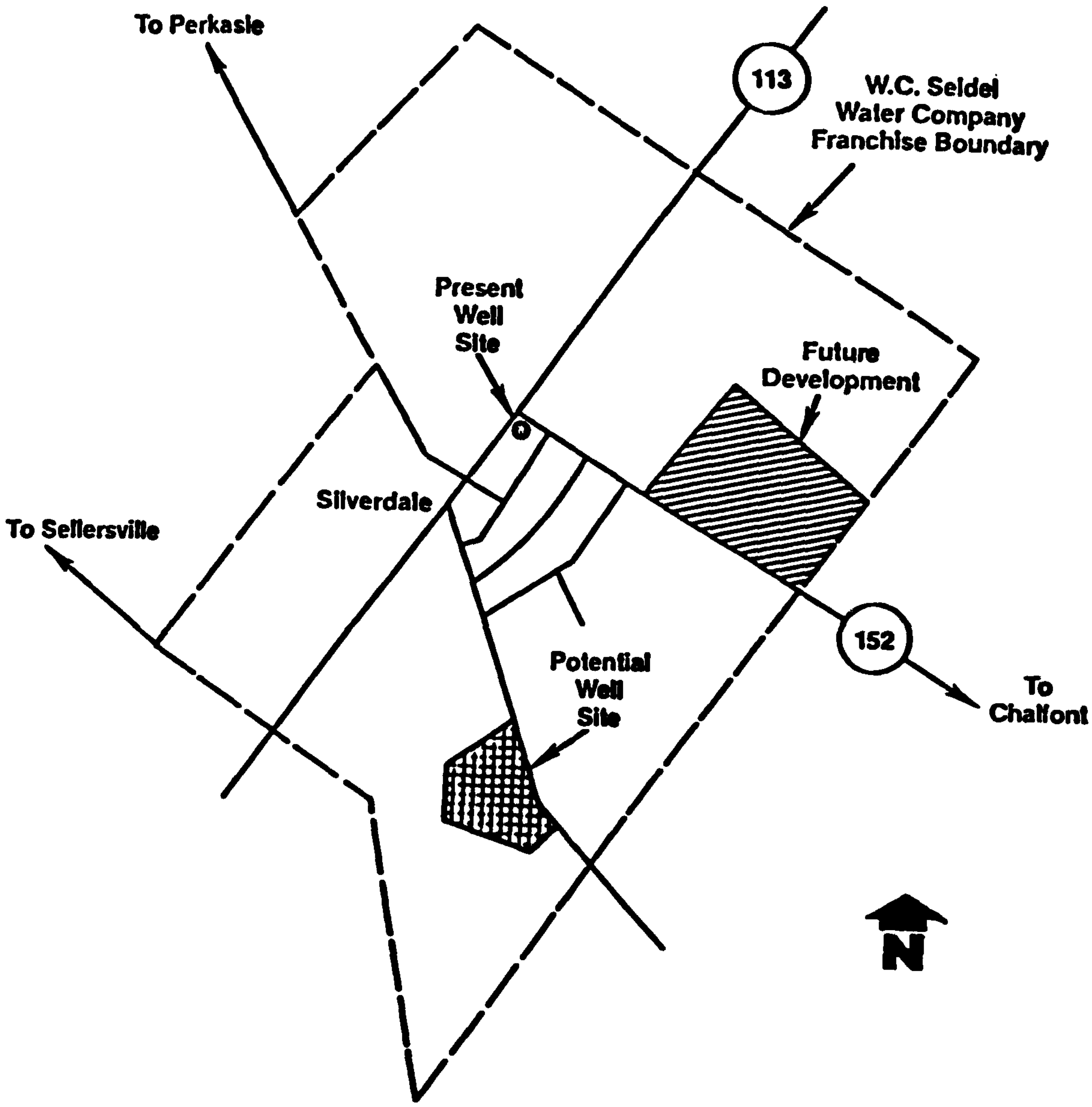

Figure 2. Silverdale test case.

Department of Environmental Resources have shown the presence of TCE in two of the wells and in the distribution system. Concentrations of TCE in the distribution system have ranged from 8 to 74 ppb.

After discussions with the utility owner, seven alternatives were investigated to resolve the water quality problem. These consist of the addition of granular activated carbon treatment at the site of the wells, the use of packed tower aeration at the well site, using point-of-use treatment devices within the residences of the customers, developing new wells within the utility's franchise area to replace the contaminated wells, developing new wells to serve an enlarged distribution system including a planned residential community, locating more distant wells, and distributing bottled water.

The results of the economic analysis of these alternatives are presented in Table 3. Shown for each of the options considered is a total present worth for a 25-year planning period as well as the total cost in terms of dollars per thousand gallons. This unit volume cost was determined on the basis of the total flow needed to be replaced or treated by centralized unit processes and not simply the potable fraction accounted for by the point-of-use or bottled water options. An interest rate of 12 percent and inflation rate of 8 percent were assumed over the planning period.

This preliminary design analysis indicates that the use of package granular activated carbon (GAC) treatment would be cost effective. The total present worth cost would be about \$50,400 or equivalent to a cost of 55 cents per thousand gallons over the planning period. The initial construction cost would be \$19,900 with initial O&M costs of \$1,830 per year. The costs are based upon a cylindrical, pressurized, downflow steel contactor furnished with inlet and outlet nozzles, underdrain system, valves, pressure gauge, backwash pump and an initial charge of activated carbon. Preliminary design would indicate a two-foot diameter column with a hydraulic loading rate of 5.4 gpm/ft^2 and 7.5 minute contact time. O&M costs include electrical requirements, maintenance material requirements (including annual carbon replacement), and labor. Labor involves backwashing the column once per week and performing routine maintenance tasks. No allowance for laboratory monitoring is included.

TABLE 3

SILVERDALE TEST CASE

(Population = 150)

TCE CONTAMINATION
(25-Year Planning Period, 12% Interest, 8% Inflation Rate)

Alternative	Total Present Worth ($)	Cost per 1,000 Gallons ($)
1. Granular Activated Carbon Columns	50,400	.55
2. New Wells with Expanded System (Existing System's Share)	61,000	.66
3. Aeration	91,800	.98
4. New Wells at Proposed Site	134,000	1.47
5. Point-of-Use Treatment	181,000	1.99
6. New Wells (5 Miles)	261,000	2.86
7. Bottled Water Distribution	271,000	2.97

The expansion of the service area to include new residential customers and utilizing new wells in or near that development to replace the contaminated wells as well as serving the new development, is also shown to be competitive. In considering this option, only those costs related to supplying an equal volume to the existing system as that which would be lost by abandoning the contaminated wells were used for comparison with the other alternatives.

It is assumed that the costs involved in providing water solely to the new community will be recovered through the housing costs or the rates established for the new customers. The total present worth for replacing the present wells from the new wells in an expanded system would be $61,000 or 66 cents per thousand gallons. This was developed by estimating the total cost of developing new wells, storage, and the pumping cost for the expanded system (assuming a total of 600 customers at the start of the planning period) and proportioning the existing system's share of that cost as 25% (150 customers). The total initial construction cost of this option would be $133,000 with initial O&M costs of $7,400/year. The share apportioned to the existing system was $33,000 in initial capital costs and $1,850 in initial annual O&M costs. The system provided assumes a new well with a yield of 0.144 mgd, 10" bore diameter and 400 feet deep, ground storage of 50,000 gallons and pumping costs at 6 cents/kw-hr. It should be noted that although this option is slightly more expensive than the use of GAC columns when only considering the resolution of the TCE problem, the alternative may be preferred overall as the economies of scale from the expanded water utility should produce savings in the long run operation of the overall water utility.

The next alternative is the use of a packed bed aeration system to treat the TCE containing water before distribution. The aeration unit process itself is less expensive than GAC treatment but requires repumping of the water after passage through the tower. Because of the location of the wells at the lowest elevation of the distribution system, this requirement for pumping the water a second time make this alternative more expensive than the two previous alternatives. Limited experience with the process indicates removal of between 40 and 90 percent of volatile organic compounds. The total present worth of this alternative would be about $91,800 or 98 cents per thousand gallons over the planning period. The initial construction cost would be $27,800 with initial O&M costs of $2,890 per year. The costs are based upon a preassembled 22 foot rectangular aeration tower with 16 feet of PVC media, an electrically driven, induced-draft fan, and repumping of the treated water to distribution.

The location of new wells within the present franchise area to replace the affected wells is another possible alternative. The costs of developing new wells at a proposed future well site and piping 3,850 feet to the present distribution system were developed. The total present worth of this alternative is $134,000 with that cost equivalent to $1.47 per thousand gallons. The initial capital investment required would be $35,000 with initial O&M costs of $5,850 per year.

Because there have been no well tests conducted within the existing franchise area for well development, it is not certain that wells with sufficient yield to replace the existing affected wells could be located. For this reason, an alternative was considered assuming that wells would have to be located five miles from the present distribution system. The additional piping and pumping required would be the primary difference between this and the previous alternative. The total present worth of this option would be $261,000 or $2.86 per thousand gallons. The initial capital costs would be $170,000 with initial O&M costs of $6,140 per year. This shows that for considering alternate well sources, the distance of any potential well site from the present distribution system is a major factor in the total cost.

The use of point-of-use treatment devices for TCE removal within the customers homes was also examined. This would involve the installation of a GAC cartridge device under the kitchen sink, adding an extra faucet, and instructing all customers to use that source for drinking and cooking purposes. Cartridges were assumed to require replacement twice per year. The total present worth of this alternative is $181,000 per thousand gallons. The initial capital investment of $14,600 is the lowest of any of the options investigated, however the requirement of cartridge replacement makes this a very O&M intensive alternative with initial O&M costs of $7,430 per year. Thus, this option would be relatively inexpensive to initiate but its recurring costs would be considerable. In addition, the questions of liability, who would be responsible for maintenance, and monitoring requirements for in-home treatment devices would have to be resolved.

The final alternative evaluated was the distribution of bottled water. This option considered the door-to-door delivery of bottled water on a twice-weekly basis in order to provide a volume equivalent to one gallon per person per day for drinking and cooking. It is assumed that the responsibility would be contracted to a bottler but the costs presented represent those of transporting and delivering the bottled water and do not include profit. The total present worth of this alternative is $271,000 and the cost per thousand gallons is $2.97. This would represent an initial contract for bottled water distribution of $13,100 per year.

SUMMARY

The WATer MANagement (WATMAN) model has been expanded in this study so as to cover both large and small water systems. It is a particularly versatile tool for estimating preliminary design costs for small water systems as data appropriate to small systems and alternative technology, especially for bottled water distribution and point-of-use treatment, was collected and used to derive cost information. Cost functions for estimating capital and operations for small water systems have been developed and incorporated into the model. WATMAN is flexible enough to simulate almost any configurations of up to ten water systems and a variety of unit processes in order to conduct preliminary designs and economic evaluations of alternative technologies available to utilities. Several technologies considered especially appropriate to small systems were examined including alternate sources, conventional additional treatment, package plants and equipment, regionalization, and dual water systems, point-of-use treatment, and bottled water distribution.

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AD P001907

DEVELOPING COST ESTIMATING METHODS FOR SURFACE-WATER INTAKE STRUCTURES

Ken Cable and Janet S. Condra[1]

BACKGROUND

The development of water supply systems involves several phases of design. In the early stages of water supply studies, it is desirable to rapidly prepare preliminary designs and cost estimates for a large array of alternative designs. While the Environmental Protection Agency (Gumerman, et al., 1979) and the U.S. Army Engineer, Waterways Experiment Station (WES) (U.S. Army Engineers, Office of the Chief of Engineers, 1979) have developed procedures which are easily used and supply reasonable cost estimates and design data for many water supply facilities, available procedures for surface-water intake cost estimating are not quite as advanced.

Surface-water intake structure design is highly site-specific, and hence the costs depend on a large number of variables. A simple correlation of intake cost with design flow or total system cost will not provide accurate estimates due to the variability in designs for the intake system. Therefore, a more complex procedure involving several site-specific characteristics is needed to estimate reasonable costs for surface-water intakes. Presented in this paper are: 1) existing approaches for developing planning level cost estimates for intake structures and/or raw water pumping facilities, 2) the procedure developed by the authors for preparing quick estimates 3) comparisons of costs generated by this method with costs of actual projects, and 4) implications of the estimating procedure.

APPROACHES

Several procedures for estimating intake structure and raw water pumping costs have been presented in the literature. Some of these methods limit their accuracy by correlating cost with only one design parameter. Hinomoto (1977) developed capital cost functions for a surface water treatment plant and its components by using the following power function:

$$C = aQ^b \qquad (1)$$

where C is the investment cost in 1972 dollars, Q is the design capacity in million gallons per day, and a and b are constants. Hinomoto uses cost data from nine plants to determine the a and b parameters for raw water intakes and pumping stations. Raw water intake and pumping includes the pump station building and equipment and the intake and screens. For flow rates of 1 to 12 mgd, he suggests using 6800 for a and 1.513 for b. A value of 1.513 for b is somewhat surprising since it indicates a diseconomy of scale in construction of surface-water intakes. A likely reason for Hinomoto's high b value is that a few of the larger intakes were considerably more elaborate than the small ones.

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Koenig (1966) presents several methods for including a surface-water intake in cost estimates for water supply systems. Two methods represent the intake cost as part of other component costs in the water supply system (i.e., a pumping plant or the outlet works of a reservoir). Koenig suggests that intake structure cost is roughly the same as an additional 2.5 miles of pipeline for each mile of intake pipe distance.

In the early planning stages of a water supply system, the cost for withdrawing water from the water source is sometimes represented as only the cost of the raw water pumping facility. Day, et al. (1979) suggested that raw water pumping cost be calculated as 9 percent of the total capital cost of a surface-water treatment facility. Gumerman, et al. (1979) developed cost functions for raw water pumping facilities which include total dynamic head and flow. The cost of the pumping facility includes the pumps, valves, piping, electrical equipment and instrumentation. The authors note that wet well and housing requirements are highly variable from location to location and, therefore, do not include them in the cost of the facility. They presented curves for construction and operation and maintenance costs for pumping at a 30- and 100-foot total dynamic head.

The dependence of intake cost on several site-specific variables is noted in an engineering and design manual prepared by the Office of the Chief of Engineers for estimating costs of sanitary facilities at recreational areas (Office of the Chief of Engineers, 1983). The authors note that surface-water intake cost depends on flow, distance of intake from shore, depth of water, number of intake ports, and distance from shore to treatment plant. Intake system cost for small systems (including the intake structure, pumps and pipe-to-shore) is determined by the following equation:

$$C_{in} = 1280\ KQ^{0.26} \qquad (2)$$

where

Q = maximum flow rate (10,000 to 1,000,000 gpd)

K = the ratio of the current ENR construction cost index to the January 1980 ENR construction cost index

Correlations of intake system cost with a limited number of design parameters (e.g., design, flow, or water depth) offer a less than desirable accuracy in estimating costs over the wide range of designs that exist. Since many individual cost items make up the cost of an intake structure, it may be possible to correlate each item with one or more design parameters for that item and sum the costs for a total cost. The remainder of this paper describes a procedure to calculate costs for surface-water intake structures and pumping facilities. The intake system cost functions consider a variety of site-specific variables.

This procedure has been included as a module in the Corps of Engineers MAPS (Methodology for Areawide Planning Studies) (Office of the Chief of Engineers, 1980) computer program and will be referred to as the MAPS procedure for determining cost estimates for surface-water intakes in the remainder of the paper. MAPS is a multipurpose program, regularly used by engineers and planners for the preliminary design and cost estimating for a number of water supply system components (e.g., pipelines, storage tanks, pumping stations, water treatment plants). The MAPS program can be used to generate design and cost data for a large number of alternatives and include the total cost of constructing and operating the water supply facility.

ESTIMATING PROCEDURE

The procedure used in the intake structure module in MAPS is illustrated by the schematic in Figure 1 which shows how the individual component costs are combined to determine average annual cost. Surface-water intake structures have many different designs. The MAPS procedure will calculate capital cost and operation and maintenance cost for two types of intake structures: 1) submerged and 2) exposed tower. The choice of design is dependent on several site-specific variables. The primary difference between these two types of intakes is the construction of a cofferdam (required for the exposed tower) versus onshore excavation for a pump station (usually associated with a submerged intake).

The construction costs for an intake structure can be divided into several components. A cost estimate for each component may be determined by correlating each component with its principle design parameters. The sum of component costs will represent the total cost for the intake system. The submerged intake system may include the following: 1) submerged crib, 2) subaqueous pipe, 3) pumps, 4) pump housing, and 5) wet well. The exposed tower intake system may include the following: 1) tower, 2) cofferdam, 3) pilings for poor subsoil conditions, 4) bridge (vehicle or pedestrian), 5) pumps, and 6) pipe (bridge-supported or subaqueous). The cost functions developed for each component were based on cost estimates for submerged and exposed tower intakes prepared by the firm of CH_2M-Hill for typical designs and were verified using data from the firm's files.

The total operation and maintenance cost, including equipment replacement, labor, and energy, is roughly the same for exposed tower and submerged intakes. Equipment replacement is estimated as 0.5 percent of the capital cost. Annual labor cost is determined for both general maintenance and major repair. Average annual energy costs are calculated from a unit price of energy, pump efficiency, average head, and average flow.

VERIFICATION

To verify the MAPS cost estimating procedure for surface-water intakes, cost estimates were compared with actual project costs for 10 surface-water intake projects. These intake systems represented a wide array of designs. Pipeline lengths for the eight exposed tower intakes ranged from 46 to 3000 ft and bridge lengths ranged from 15 to 500 ft. The head delivered by the pumps ranged from 10 to 112 ft and tower height ranged from 24 to 53 ft. Five exposed tower projects had poor subsoil conditions and thus required pilings. The two submerged intake projects had a pipe length of 200 and 250 ft and a head of 250 and 285 ft. An intake without a crib or tower had a pipe length of 2000 ft and a head delivered by the pumps of 600 ft.

The average absolute value percent error in comparing the MAPS and actual costs was 13.9 with a range of 1.3 to 26.8 percent. Figure 2 shows how the MAPS cost estimates vary from the actual costs. Five of the estimates were higher and five were lower than the actual costs. As shown in Figure 3, cost differences did not appear to vary significantly with a change in flow capacity. Figure 3 also indicates that correlating construction cost with flow alone could not produce accurate estimates.

IMPLICATIONS

A sensitivity analysis was performed to determine the effect on costs when increasing various parameters by 50 and 100 percent for both submerged and exposed tower intakes. The results, illustrated in Figure 4, indicate that the most significant parameters causing an increase in the exposed tower intake cost were flow and tower height. Increasing the bridge length, pipeline length, and the maximum head delivered by the pumps had a much smaller effect on the total construction cost. As shown in Figure 5, a 50 to 100 percent increase in flow and wet well depth had the most significant effect on the total cost of submerged intakes. If cost is represented by

$$C = aX^{b} \tag{3}$$

where

- C = cost
- a = constant
- X = parameter value
- b = economy of scale factor

the sensitivity of the parameter to change can be determined by the value of the economy of scale factor, b. The values of b for each parameter are given in Table 1. These values are considerably different from Hinomoto's value of 1.51 indicating that economy of scale does exist in surface-water intakes.

A parameter is considered more sensitive to change as the b value increases. These values concur with Figures 4 and 5 indicating that flow and tower height have the most significant effect on exposed tower intake costs and that flow and wet well depth have the most significant effect on submerged intake costs.

Construction cost estimates were determined by the MAPS, Gumerman, et al. (EPA), and Hinomoto procedures for a range of design capacities and these results are compared in Figure 6. The values predicted by Gumerman, et al. include the costs for pumps, valves, piping, electrical equipment and instrumentation but do not include pump housing and wet well costs, and therefore, are expected to be lower than the MAPS predicted costs. The MAPS and EPA curves exhibit basically the same shape. The construction costs predicted by Hinomoto for flows from 1 to 12 mgd include the pump station building and equipment and the raw water intake and screens. For the same range of flow, submerged intake construction costs predicted by MAPS are higher than Hinomoto's estimates. Hinomoto's procedure yields inordinately low costs for small intakes, indicating the designs of these intakes may be considerably simpler than the MAPS or EPA designs.

Unit costs were determined for submerged and exposed tower intakes for flows ranging from 10 to 100 mgd. As a basis of comparison, the wet well depth in the pump station for the submerged intake was set equal to the tower height of the exposed tower intake. A 500-foot long submerged pipeline was used for all cases. The curves of Figure 7 show that the unit price, in cents per 1000 gallons, for exposed tower intake systems was consistently higher than submerged intake systems. The higher costs for exposed towers can be attributed to the construction of the cofferdam. The differences in cost become smaller with increasing flow.

Table 1

Economy of Scale Factor, b

Parameter* (X)	b
Submerged	
Flow (10 - 20, mgd)	0.67
Wet Well Depth (35 - 70, ft)	0.47
Maximum Head (30 - 60, ft)	0.22
Pipeline Length (100 - 200, ft)	0.18
Exposed Tower	
Flow (24 - 48, mgd)	0.54
Tower Height (40 - 80, ft)	0.40
Bridge Length-Pipeline Length (100 - 200, ft)	0.26
Maximum Head (70 - 140, ft)	0.21

* Based on a change in parameter of 100%

Bar diagrams are presented in Figures 8 and 9 to illustrate the percentage of total construction cost for various cost components of intake systems. The exposed tower intake example included a vehicle bridge and bridge-supported pipeline of 250 ft, poor soil conditions and no onshore pump station (i.e., pumps in tower). Pipe diameter and number of pumps were calculated as a function of flow. The head delivered by the pumps was 30 ft. The bar diagrams of Figure 8 show the percentage of total construction cost for the exposed tower intake structure, pump mechanical and electrical equipment, pipeline, and bridge costs. Two values each of flow and tower height were used to evaluate how the change would effect the cost percentages for each component. Component costs indicate that bridge cost remains constant while intake structure, pipeline and pumping equipment costs increase significantly with increasing intake size. The bar diagrams of Figure 9 illustrate the percentage of total construction cost for the submerged intake structure, pumping equipment, the onshore pump station structure and pipeline costs. Again two parameters (i.e., flow, pipeline length) were varied to evaluate the effect on the cost percentage of each component. The costs used to develop Figure 9 show that as size increases submerged intake and pumping station structure costs remain roughly constant, while pumping equipment and pipeline costs increase significantly.

Submerged intake operation and maintenance (O&M) costs for flows ranging from 1 to 100 mgd, as predicted by MAPS, were compared to O&M estimates for raw water pumping facilities predicted by Gumerman, et al. The MAPS estimates concur with those predicted by Gumerman, et al., as indicated in Figure 10. The range of O&M values are very close for average flows from 10 to 100 mgd.

SUMMARY

The computerized intake structure cost estimating procedure presented in this paper can generate reasonably accurate planning level cost estimates with minimal effort. The consideration of several site-specific variables allows for a wider application of the estimation procedure than previously developed procedures.

ACKNOWLEDGMENTS

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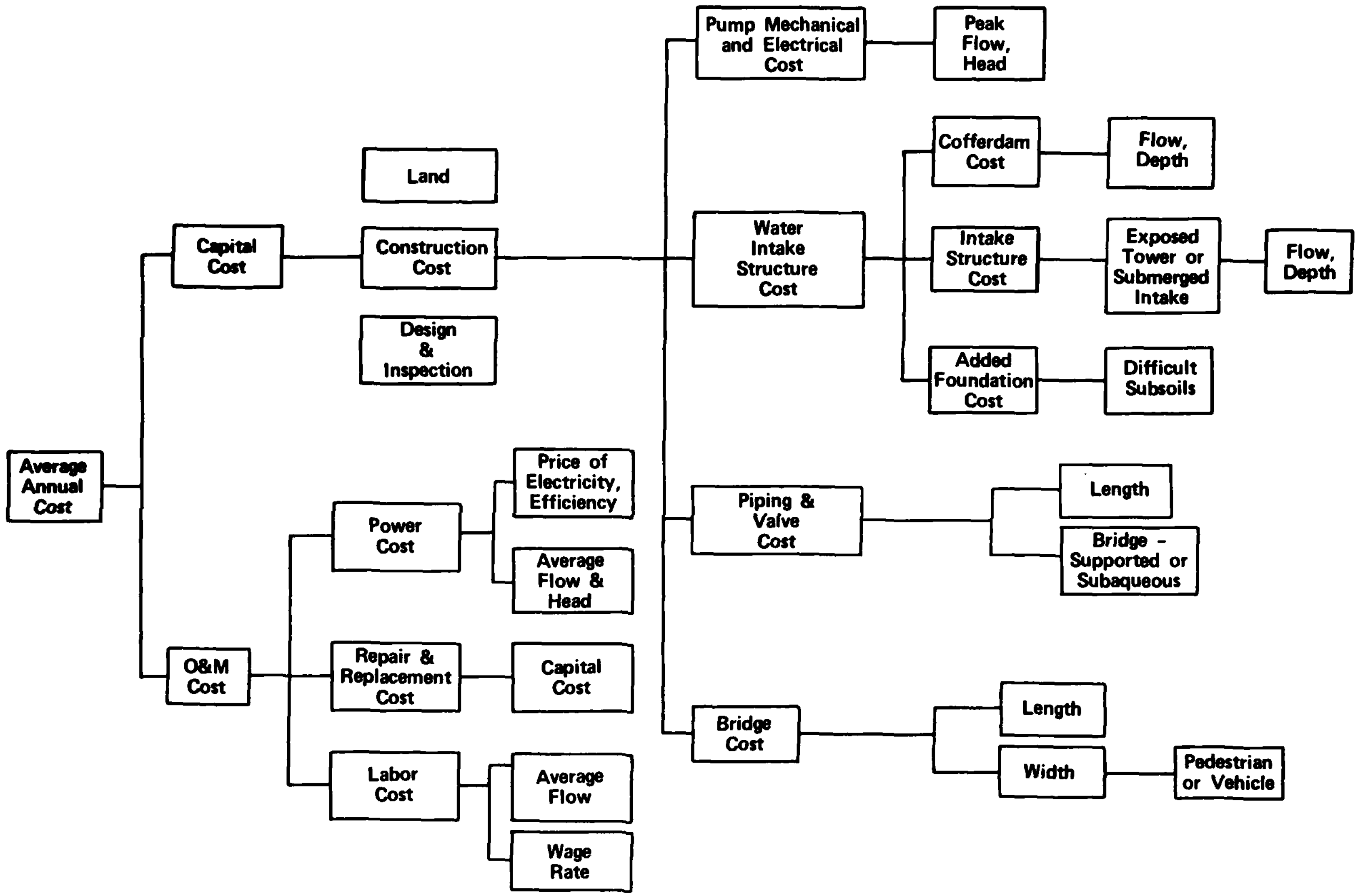

Figure 1. Surface-Water Intake System Cost Schematic

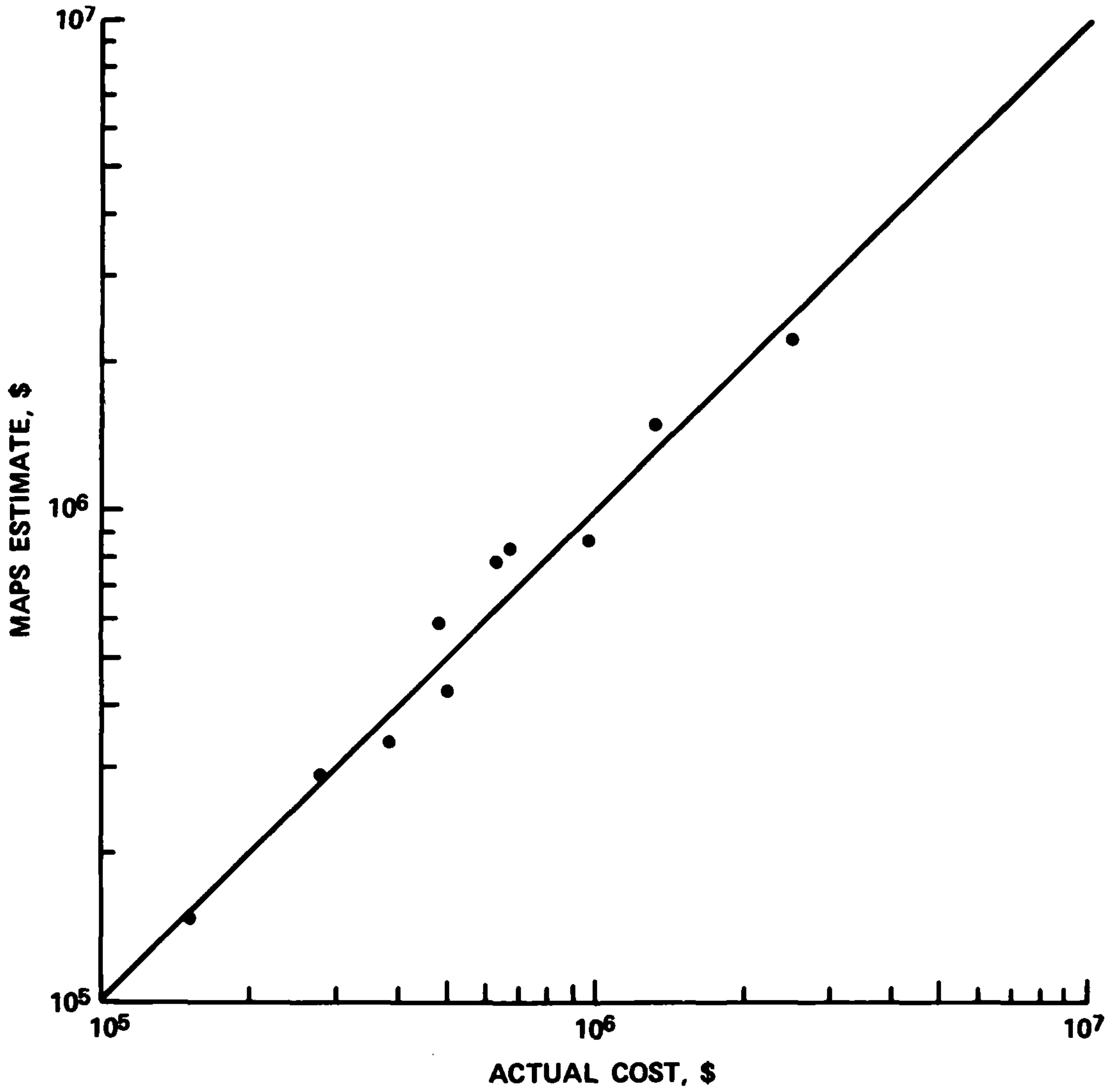

Figure 2. Comparison of MAPS Estimate With Actual Costs

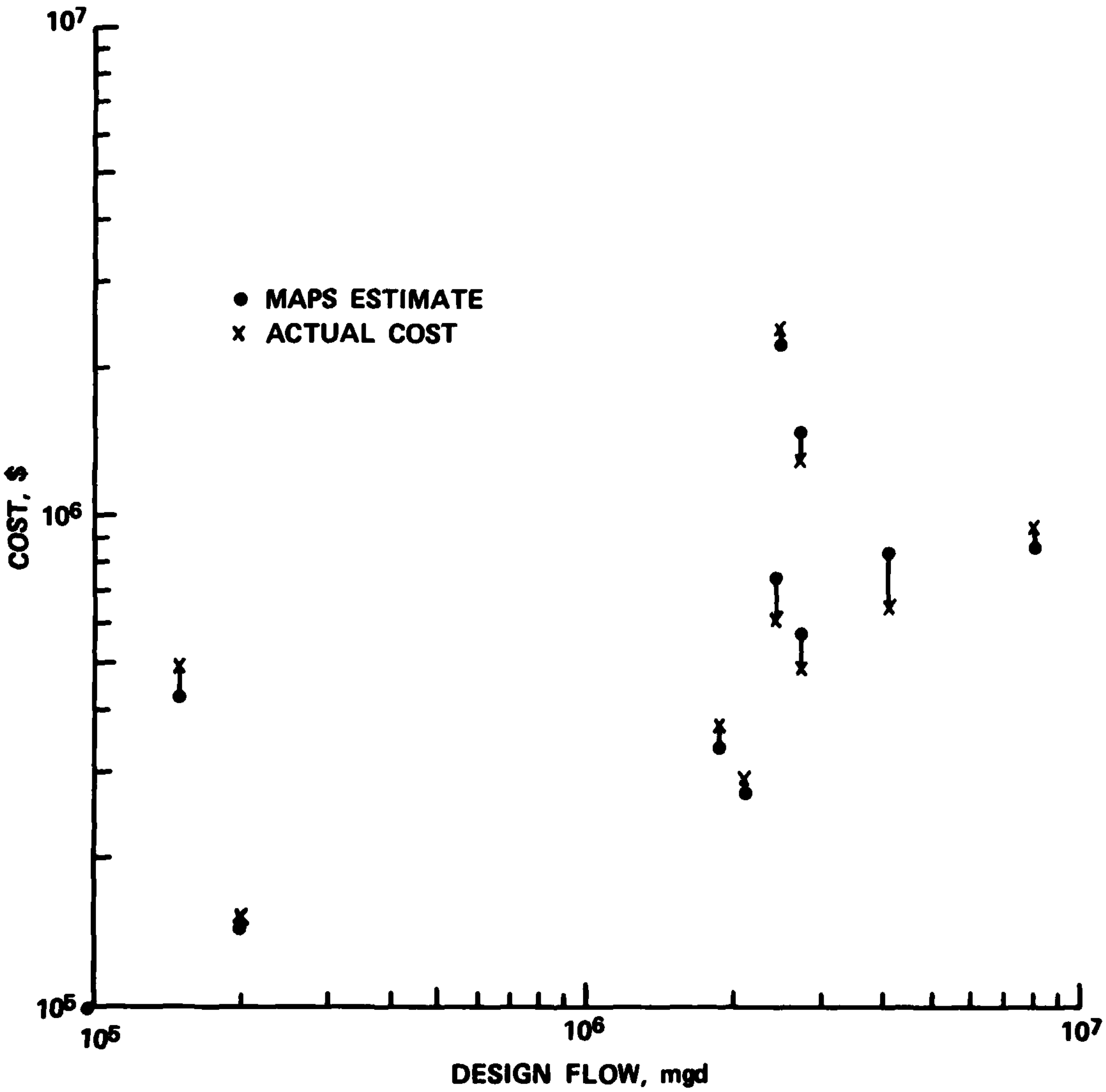

Figure 3. MAPS and Actual Cost Estimates as a Function of Design Flow

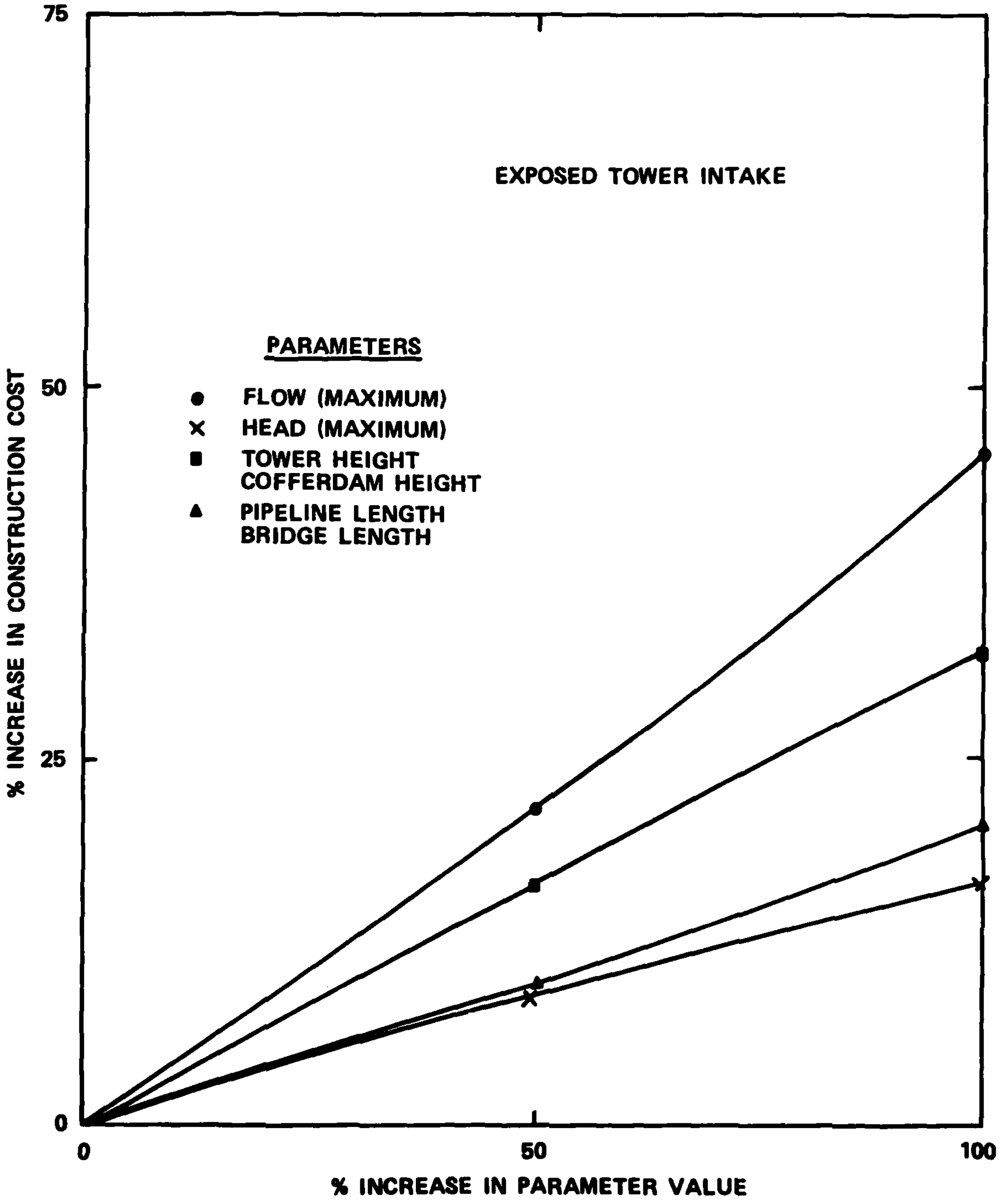

Figure 4. Sensitivity Analysis of Construction Cost - Exposed Tower Intakes

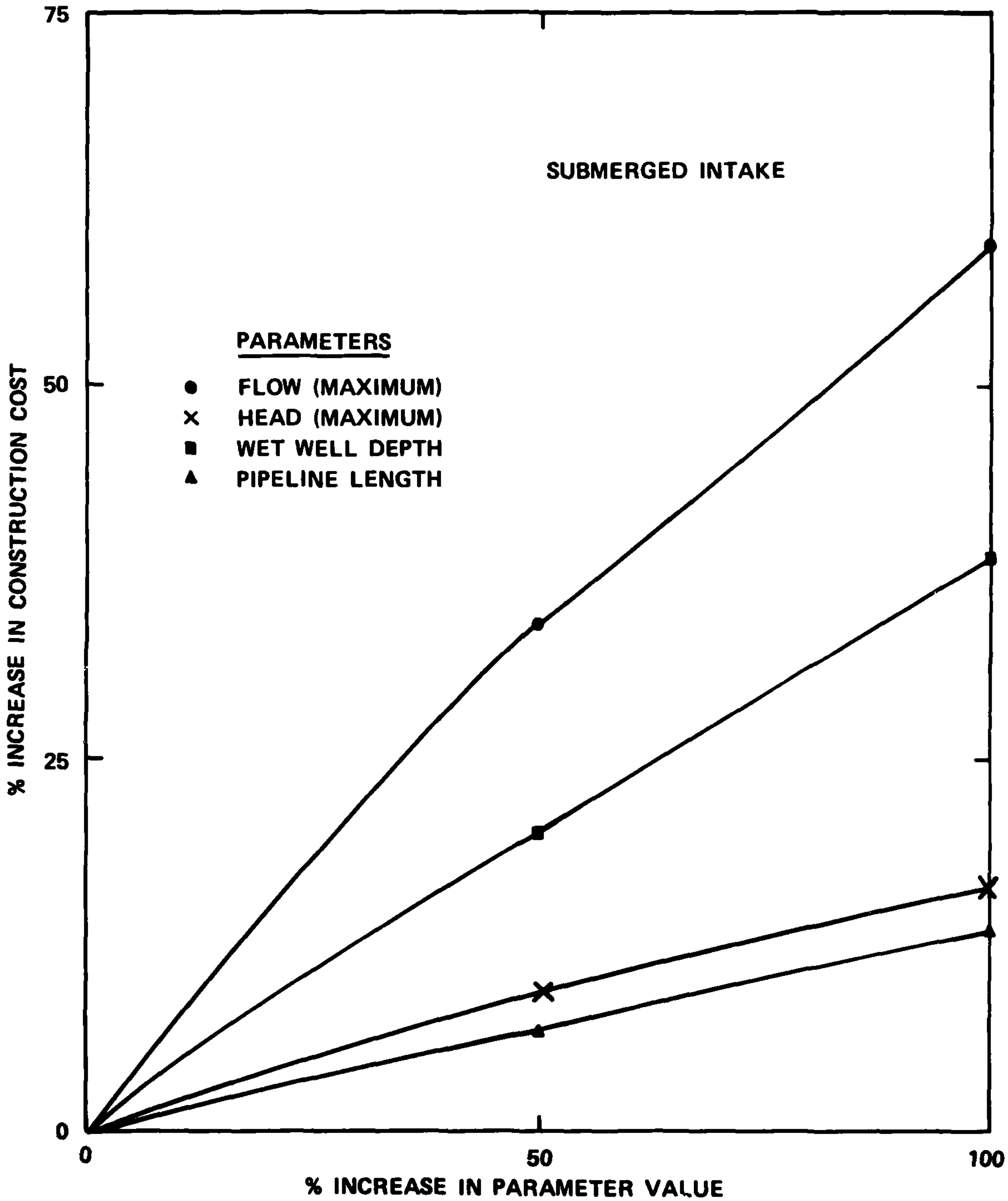

Figure 5. Sensitivity Analysis of Construction Cost - Submerge Intakes

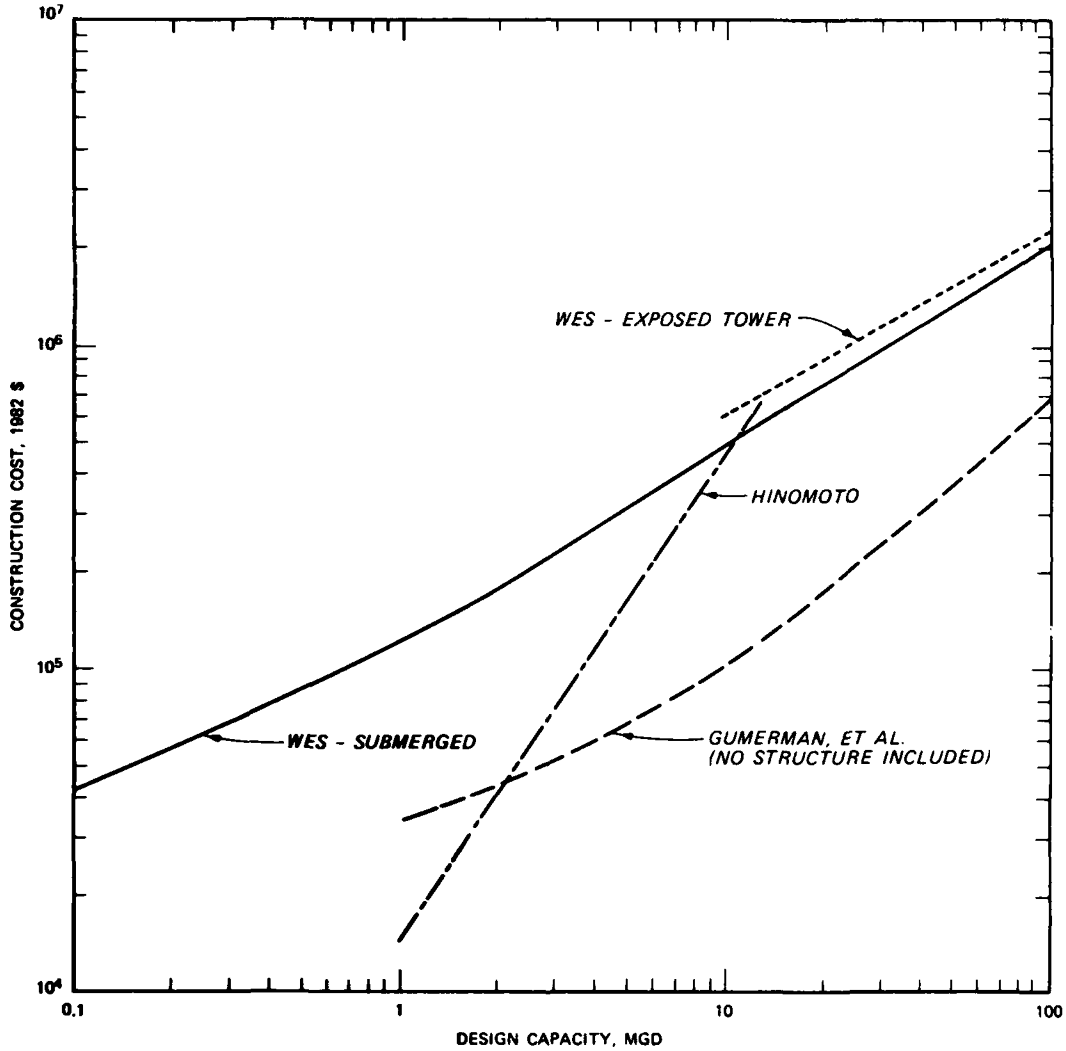

Figure 6. A Comparison of Cost Estimating Procedures

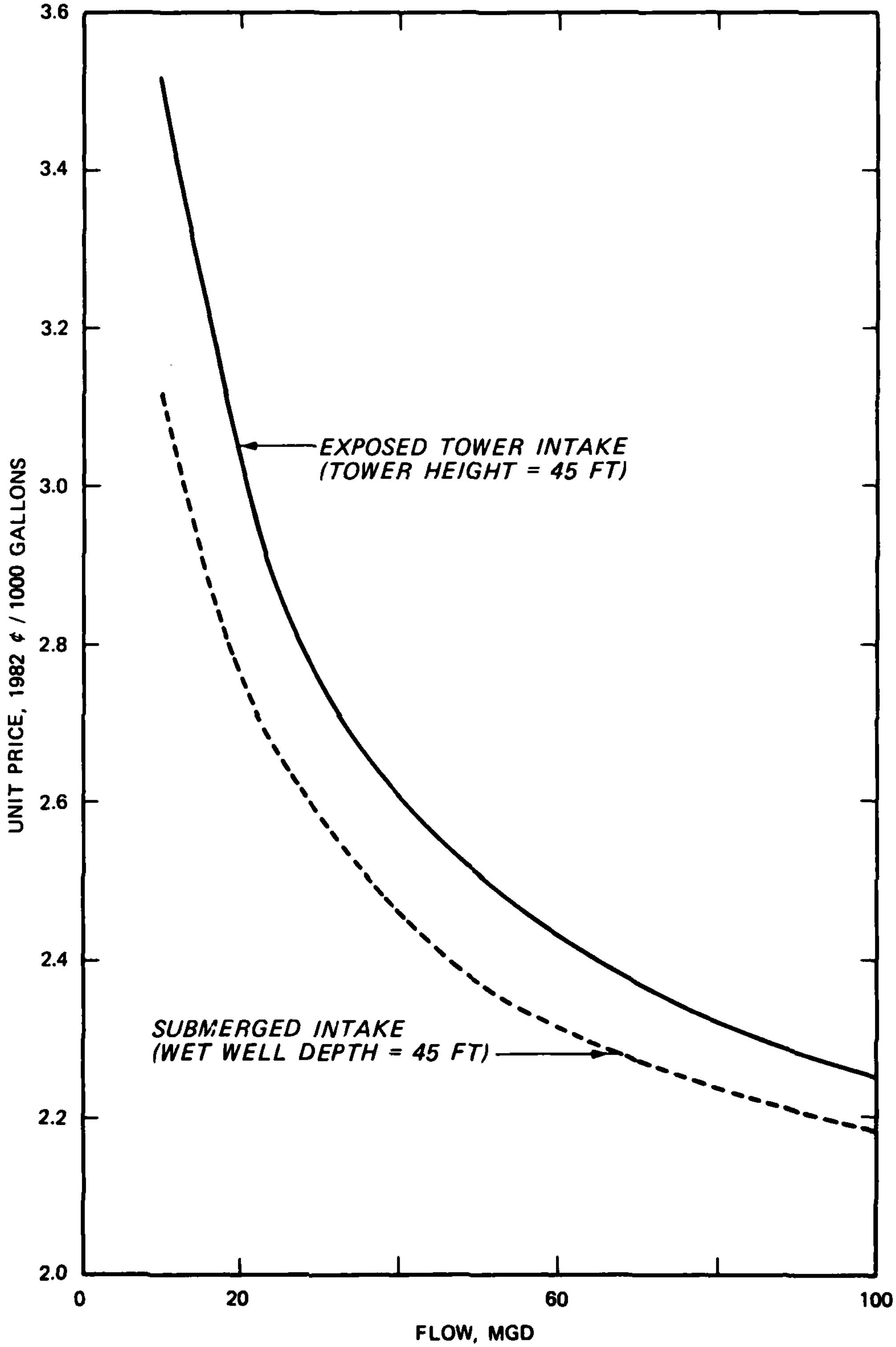

Figure 7. Unit Price Comparison of Submerged and Exposed Tower Intakes

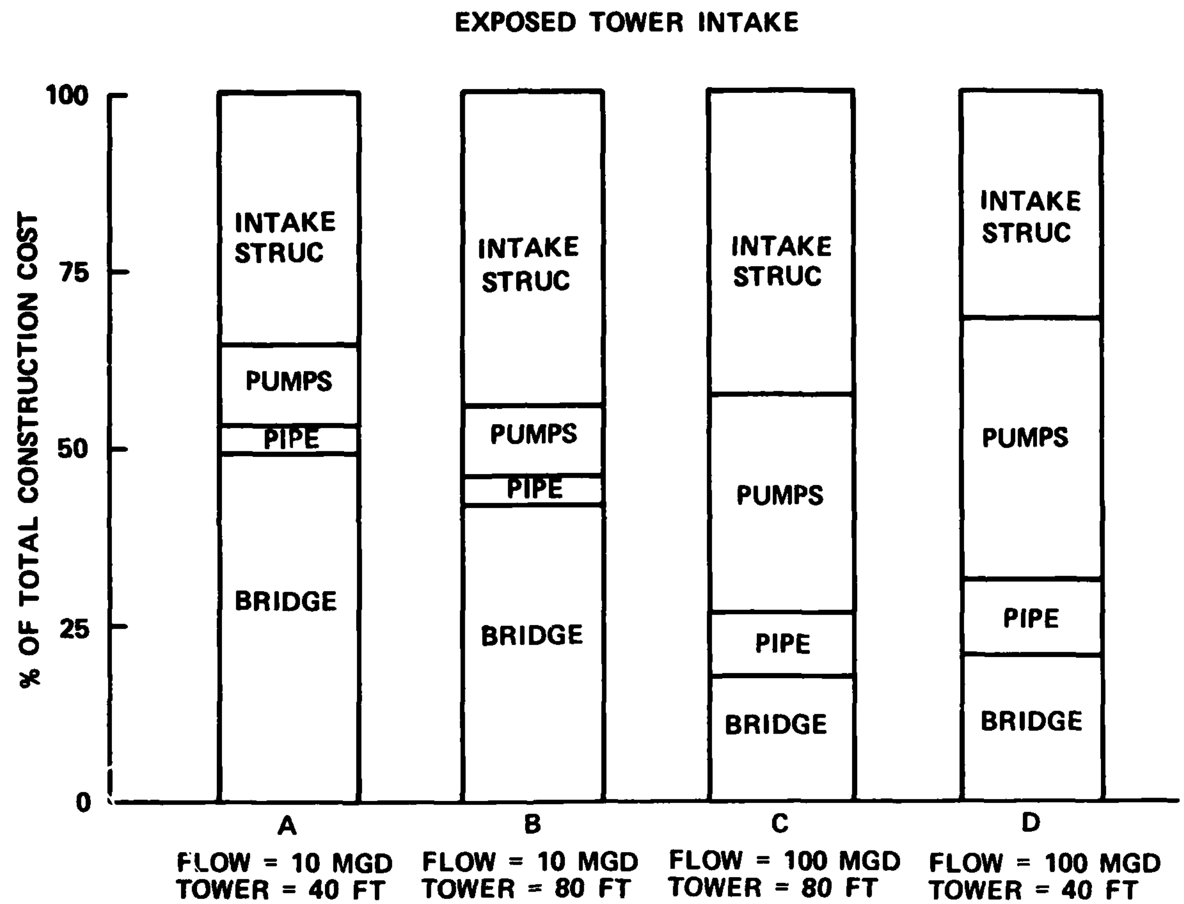

Figure 8. Component Costs as a Percentage of Total Construction Cost - Exposed Tower Intake

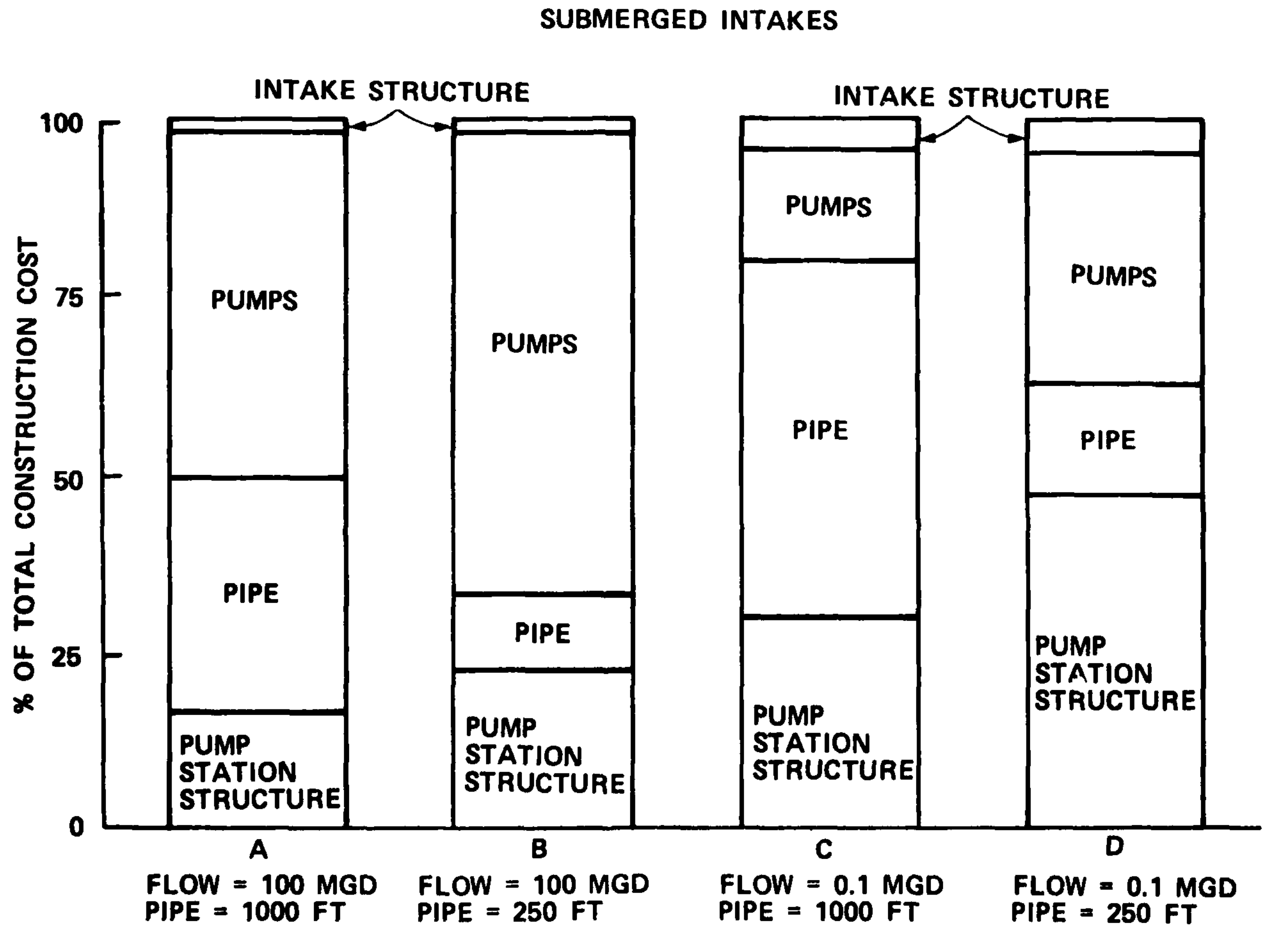

Figure 9. Component Costs as a Percentage of Total Construction Cost - Submerged Intake

AD-A134 010 PROCEEDINGS OF SYMPOSIUM ON COST ESTIMATING FOR WATER SUPPLY PLANNING STU..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS ENVIR.. T M WALSKI

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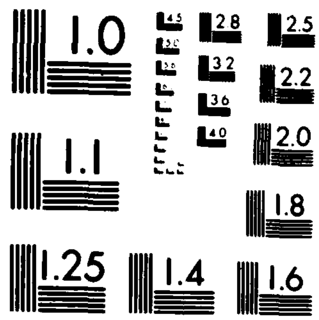


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS 1963 A

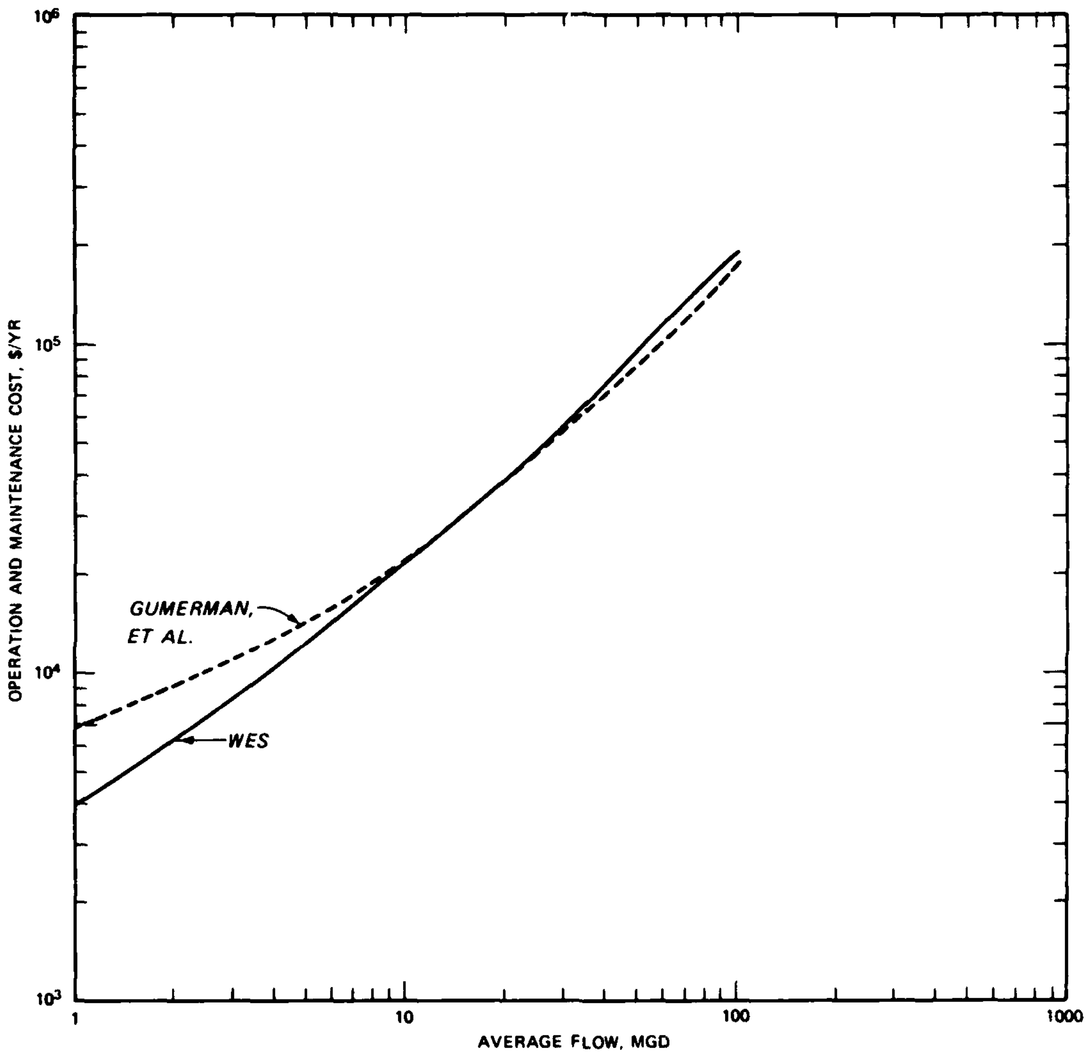


Figure 10. A Comparison of Operation and Maintenance Cost Predictions Method

AD P001908

APPLICATION OF PARAMETRIC ANALYSIS TO WATER SUPPLY

BY

H. KEITH BURBRIDGE

The theme of this conference Water Supply - The Management Challenge - *implies that there is an optimum method* by which the resources implicit in the assurance of an adequate water supply can be managed. Resources, both human and material have a direct impact on monetary resources, which are the "Bottom Line" if you will for most human creative endeavor. Decisions must be made to allocate money against some given need to provide water, and when the amount of money necessary is known, its administration must be vested in some senior decision-maker as a primary responsibility. Regardless of the source of capital, the classic sequence of cause and effect proceeds, often in a Delphi manner. First the need must be evinced, an initial estimate of the cost to fulfill that need must be made, a decision to proceed must be taken, allocation of resources must follow, and the selection of an administrator for those monetary resources must be undertaken. With the train of events completed, the administrator assembles a team of professionals equipped with diverse skills, and this team enables the Research and Development, Production/Construction process to begin, culminating in a "Water Resource On-Stream" final event. Ideally, this event should occur as the outcome of a schedule of events in sequence, and at that point, and again ideally, the project is on-time, and on-budget. Thereafter, the expenditures required to maintain the water supply/resource throughout its useful life period

should be tracked, against some estimate made for them, a priori. Today we hear much of Design To Life Cycle Cost (DTLCC), which may be defined as the cumulation of RDT&E Cost, Full Scale Development Cost, Production/Construction/ Installation Cost, Operation & Maintenance/Service Cost, and Cost to Retire the Supply at End of Useful Life. Thus, the cost picture does not end with a facility on stream, but continues throughout service life and this requires continual tracking of expenditures per period against estimates made for those expenditures.

I am not an expert in any sense on water resources and/or their costing. I do know however, that creation of new water resources involves a wide variety of plant and equipment, and a high degree of heavy construction activity in terms of buildings and facilities, as well as the distribution network, and the operations and maintenance of all these essential elements. I contend that estimates made by traditional methods for all of the activities described are difficult to produce, and are time-consuming and thus costly, for they involve appreciable amounts of human labor. Further, tracking expenditures when the facility/resource is on-line is a tedious business. The taxpayer, from whose pocket comes most of the funding to provide water supplies, hears all too many horror stories of enormous cost over-runs on all manner of new projects. Thus,

he is in a state of confusion, often angry, and not well disposed to vote for projects, even though they be necessary for his comfort and survival. I contend further, that there are better, more rapid means today for cost estimating, and tracking of expenditures. These have come about with the wide acceptance of the omni-present computer, and the understanding that there exists a wide variety of relationships among cost values, and other variables implicit in any system. Today, we have the discipline of parametric analysis as applied to cost estimating, and this discipline when executed via the computer, gives credible estimates, very rapidly, and with relatively few estimators involved. Parametrics is not intended to supplant the traditional method of "Grass Roots" estimating, there will always be a place for that. Rather it is intended to work interactively with the traditional method. Parametric analysis gives credible estimates rapidly, when comparatively little is known about the project. As the project progresses, and better values are evolved for the many project subordinate work packages, the parametric model can be calibrated against the novel values, to give up-dates, and predictions of "Should Cost", "Cost To Go", "Will Cost", and other needed information for the decision maker.

I work most of the time in the Aerospace Industry, and in particular in that sector of it concerned with Space Destined Hardware and Software. This industry has been quick to recognize the value of estimates produced parametrically, especially in the domain of development and production of complex systems. Models have been developed such as the RCA PRICE family of general case cost estimating models which deal with the production of some tangible product. This family of models is in wide use all over the Free World and is available on a lease basis. The model does not address however, the cost of "Bricks and Mortar" or large installations such as a hydro-electric generating complex, and its cost of siting. Private industry, particularly that sector involved with construction/installation of large plant and large scale excavation and production of such commodities as coal, oil, and other continuous process flow operations, has been somewhat slower in accepting the parametric process for estimation of cost. Of recent years several cost models have been developed however, which lend themselves to the needs of private industry, particularly that sector which deals with water, oil, site construction, and installation of large scale, heavy and costly, capital equipment. It is not my purpose to act as an advocate for any of these models, but rather acquaint you with what is available in the field. I have my preferences, and have used these models with considerable accuracy and success as a consulting parametrician, but my purpose is to introduce the

modeling concept as it may apply to water supply, and its use as a management tool of considerable power. To that end, I have prepared a sequence of illustrations, most of them are self explanatory. Whether or not any of you elect to use the models shown today, is not at all my concern, although I would suggest that the subject is worthy of your investigation.

Figure 1.

Illustrates the conventional estimating process. It is a true simulation process if you will, and often employs a "cast of thousands" . . . Each member of that cast is a specialist in estimating the minutiae of some aspect of the project as a whole.

Figure 2.

Conveys the message that a project underestimated against some band of value will run out of money before the true value point is reached. Management may change, and more resources must be devoted to completion of the task, with the subsequent black mark of a cost overrun. For the job that has been overestimated, that is will cost less by definition than the value of the estimate, that estimate becomes a self-fulfilling prophesy, for resources available tends toward their expenditure.

Figure 3.

Shows the two major sides of any estimate, the management perception, and that of the estimating team, each of which arrives at a value by a different path, and with a different set of variables. The reconciliation process takes time, and it is here where models can be of great use in expediting that process.

Figure 4.

Speaks for itself, and poses a question to which subsequent figures provide the answer (hopefully).

Figure 5.

Gives some definitions for the parametric process, and these have been accepted widely by industry and the U.S. DoD.

Figure 6.

This gives the flow of the parametric process. Of significance here is the use made of a valid data base of known merit, which serves as a calibration set of criteria for the parametric model. Also of significance, is an accommodation offered by most models, to run the model in reverse. An input of known cost from a similar prior project will cause the model(s) to output parameters of significance to the model(s). These form the base with which to run the model forward, to yield an estimate for the project of interest.

Figure 7.

This illustration outlines the operation of a model which is available to the cost estimating community on a lease basis. It is called ESTEK, and its operation is shown macro-scale, together with the informational process flow. Excellent results are reported for work done by this model, and several large corporations have contracted for its use. It is "User Friendly" and requires relatively little training for an operator to access it, and obtain credible estimates.

Figure 8.

Furnishes some information on a model of considerable power and sophistication, FAST-C. This is a segment or member of a family of models, created by the inventor of the RCA PRICE family of models. It is in wide use throughout the Department of Energy, the Agency responsible for funding its development, and a number of very large corporations, among them Bechtel, and Monsanto Corp. report excellent results from its use. It is a Data Empty model, in that it requires comparatively little in the way of data to cause the model to generate estimates The data base, opaque to the lessee/user, has been reduced statistically to a set of norms which represent an industrial consensus of values considered norms for the variables represented.

Figure 8A.

This figure illustrates briefly what goes on inside the FAST family of models. While it is impossible due to time constraints to enter deeply into discussion of the operation of this model, it is worthy of note that FAST-E provides estimates for processing plant and capital equipment, and FAST-C with which the E segment interacts, provides cost for preparation of the site(s) on which plant/equipment is to be installed. Again, the model is leasable, and extremely User Friendly.

Figure 9.

Here is yet another parametric model. While not at the moment leasable to a user community, it is available on a contract basis, under which the developer would prepare estimates for a customer desirous of using the model. A brief overview of its internal operation is shown, and a tabulation of the merit of its outputs. It is a data base comparison model, and requires some conditioning of a user data base provided, to obtain optimum outcomes.

In conclusion, I have used such models as these in a wide diversity of applications. The applications range through:

- Advanced Concept Naval Vessels
- Container vessels, and bulk cargo ships
- Forward Airfield preparation and construction
- Coal gasification plant, and plant siting
- Oilfield installations
- Site excavation, and Arctic facility construction
- Water purification & desalinization plant
- Sewage Treatment Plant and plant siting
- Building and Facility Maintenance & Refurbishment

for both the U.S. Government and the Commercial Sector of U.S. Industry. As a private businessman, I derive my living from the development and application of parametric models, not only for cost purposes, but also for system simulation/optimization of the discipline. I would like to leave one word with you, or rather one phrase. This is the time where the watchwords are "Adequate Performance at an Affordable Cost" . . . and those watchword apply not only to Government, but the private sector, as competition for projects becomes ever

more intense. Effective management of resources is the criterion by which every organization is judged, at least in part, and it is in this context that I recommend to your attention parametric cost estimating by use of models. The discipline is mature, although but recently come of age. Its main virtue lies in the timeliness of its results, an outcome of the speed and power of the modern computer. In short, it is an idea whose time has come. I urge again, that you give the discipline, your earnest consideration as an effective tool for you, in management of water supply, a challenge of prime import to us all.

BFH Parametrics

WATER SUPPLY

— THE MANAGEMENT CHALLENGE —

— // —

PARAMETRIC COST ESTIMATING MODELS FOR THE WATER RESOURCE DECISION MAKER

Conventional Cost Estimating Process

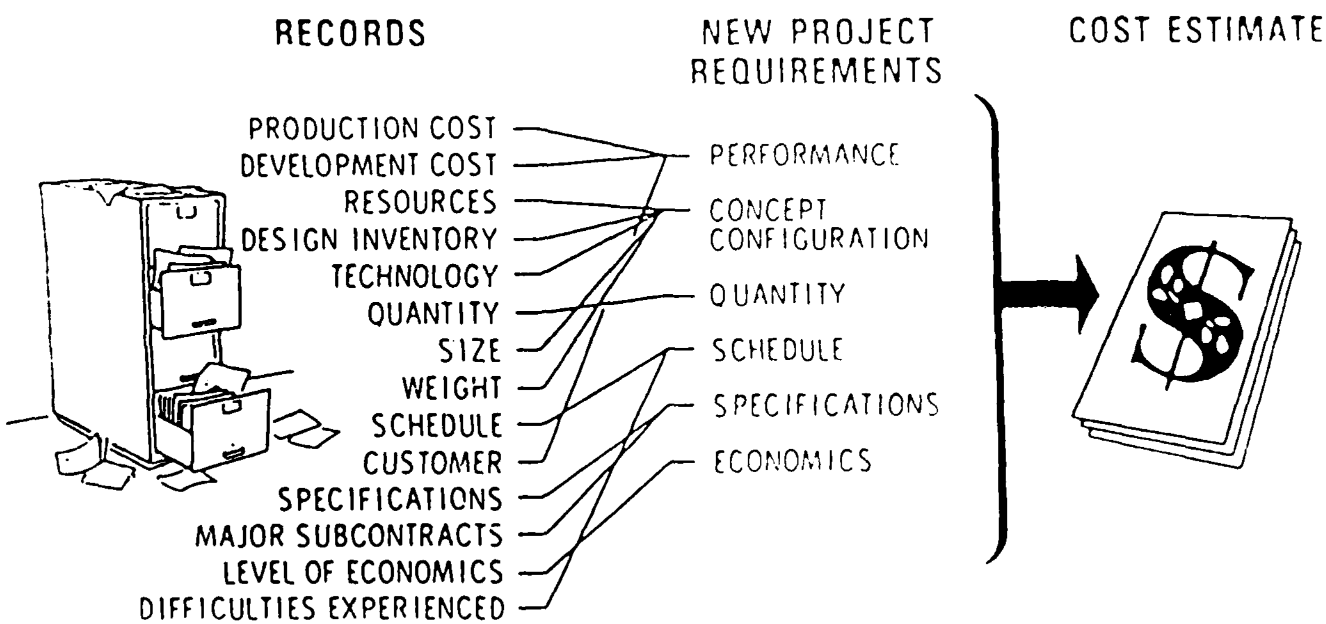
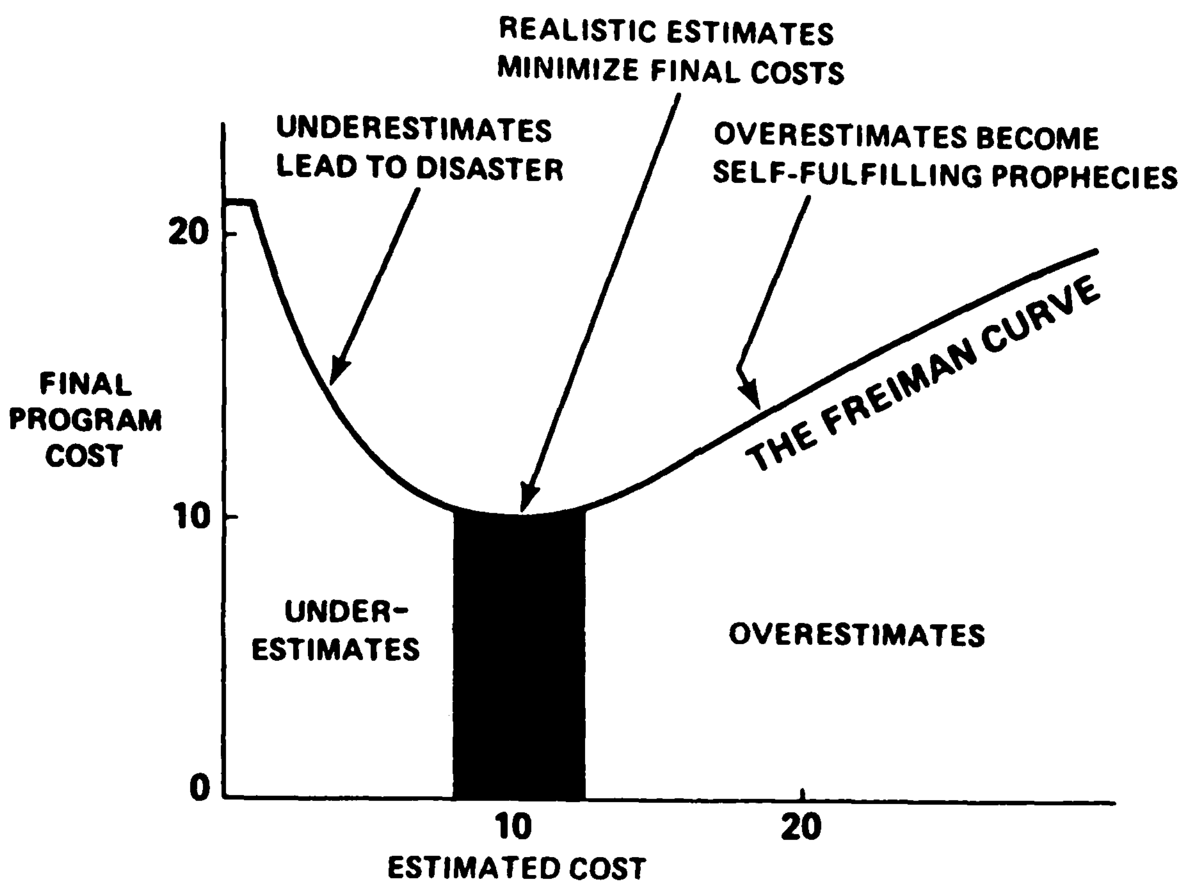
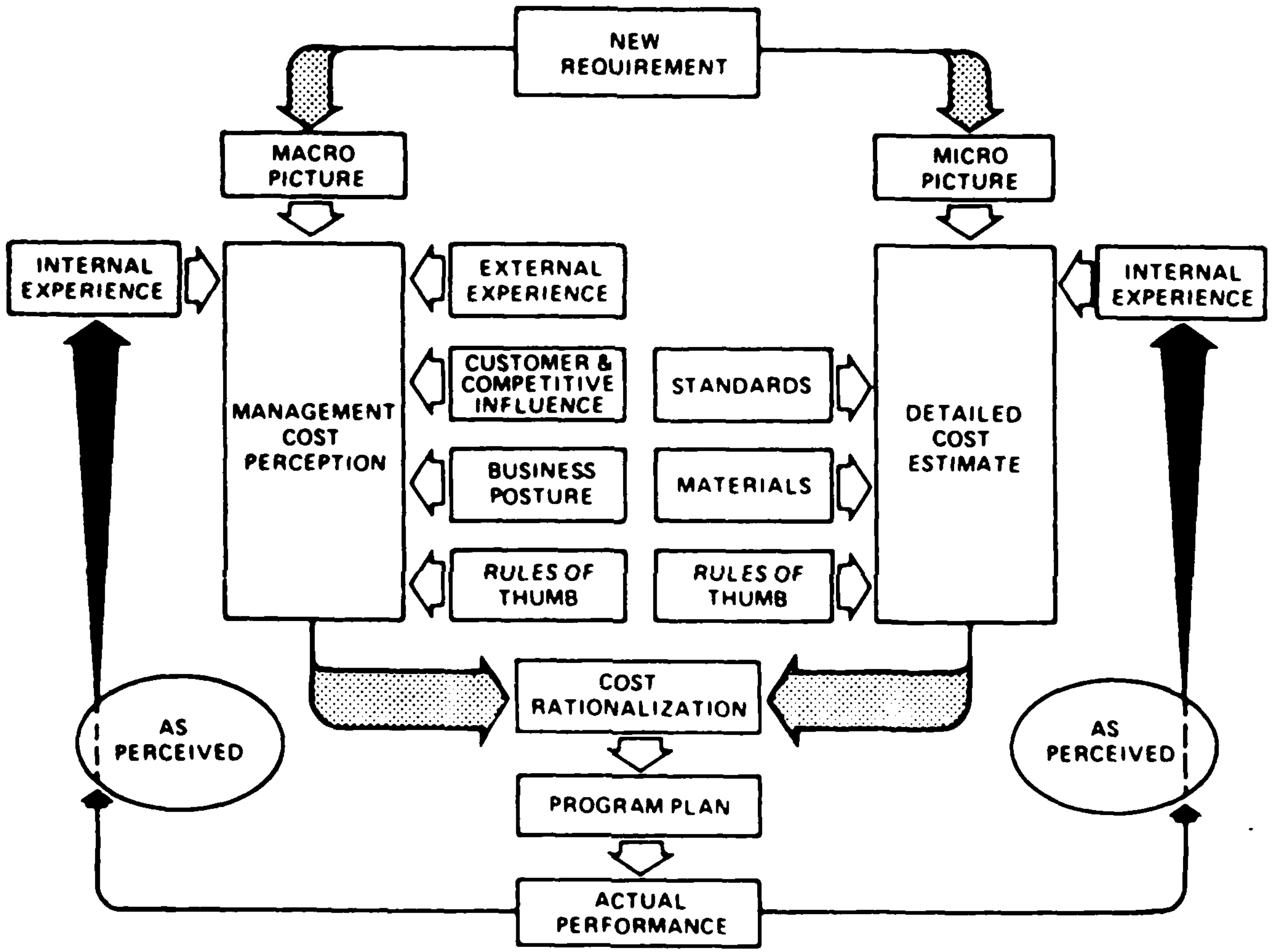


FIG.1.



The Freiman Curve

FIG. 2.



Management's Cost Perception Versus the Detailed Cost Estimate

FIG. 3.

Are There Better Ways?

FIG. 4.

* A PARAMETRIC COST MODEL *

A PARAMETRIC COST MODEL IS AN ESTIMATING SYSTEM COMPRISING COST ESTIMATING RELATIONSHIPS (CERs) AND OTHER PARAMETRIC ESTIMATING FUNCTIONS,

i.e. COST QUANTITY RELATIONSHIPS, INFLATION FACTORS,

STAFF SKILLS, SCHEDULES, ETC.

PARAMETRIC COST MODELS YIELD PRODUCT OR SERVICE COSTS AT DESIGNATED LEVELS AND MAY PROVIDE DEPARTMENTALIZED BREAKDOWN OF GENERIC COST ELEMENTS.

A PARAMETRIC MODEL PROVIDES A LOGICAL AND REPEATABLE RELATIONSHIP BETWEEN INPUT VARIABLES AND RESULTANT COSTS.

* PARAMETRIC ESTIMATING *

PARAMETRIC ESTIMATING IS A MATHEMATICAL PROCEDURE

WHERE PRODUCT OR SERVICE DESCRIPTORS DIRECTLY YIELD CONSISTENT COST INFORMATION.

FIG. 5.

Parametric Cost Analysis

THE PARAMETRIC PROCESS

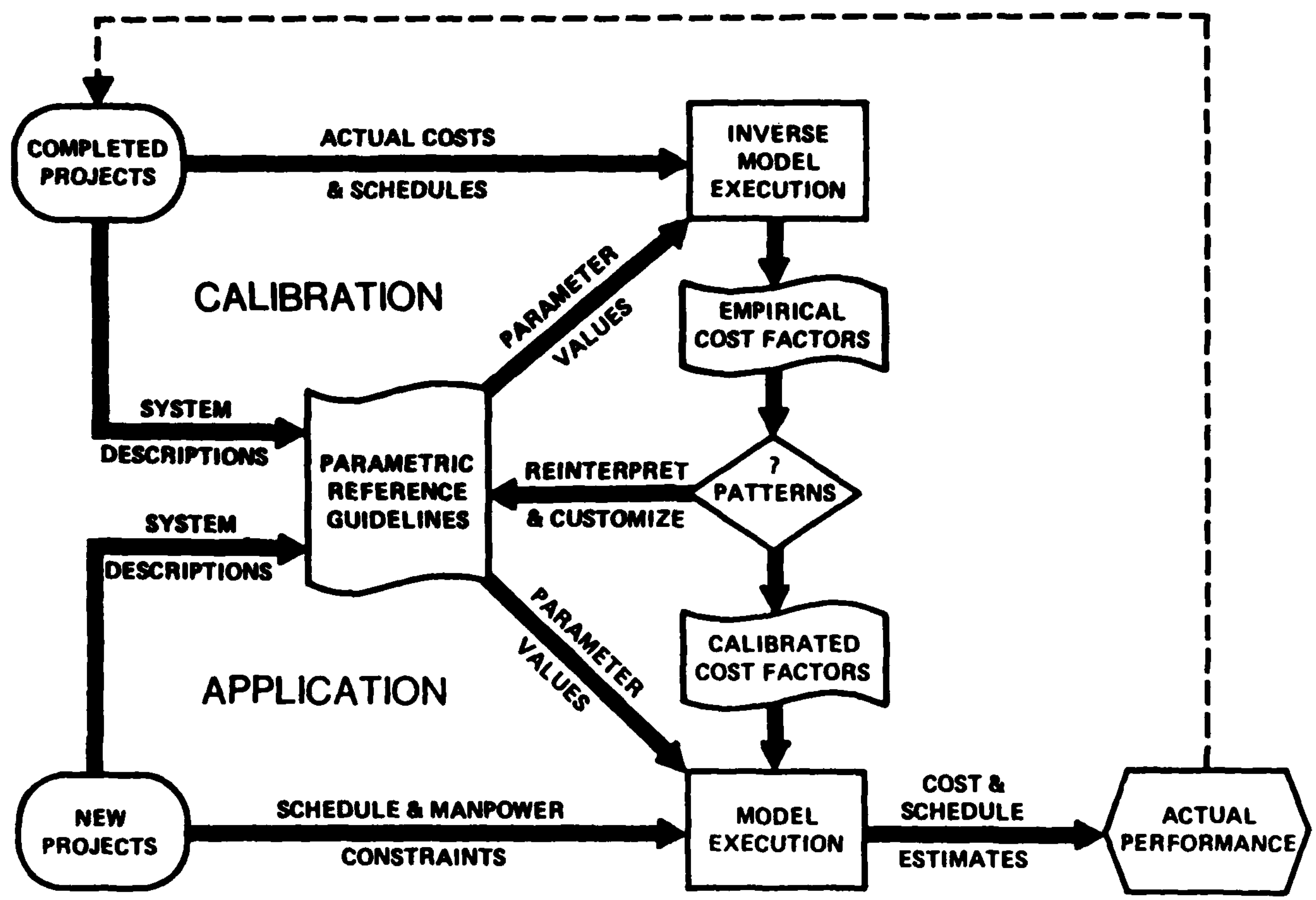


FIG. 6.

CONSTRUCTION COST ESTIMATING MODEL (LEASABLE)

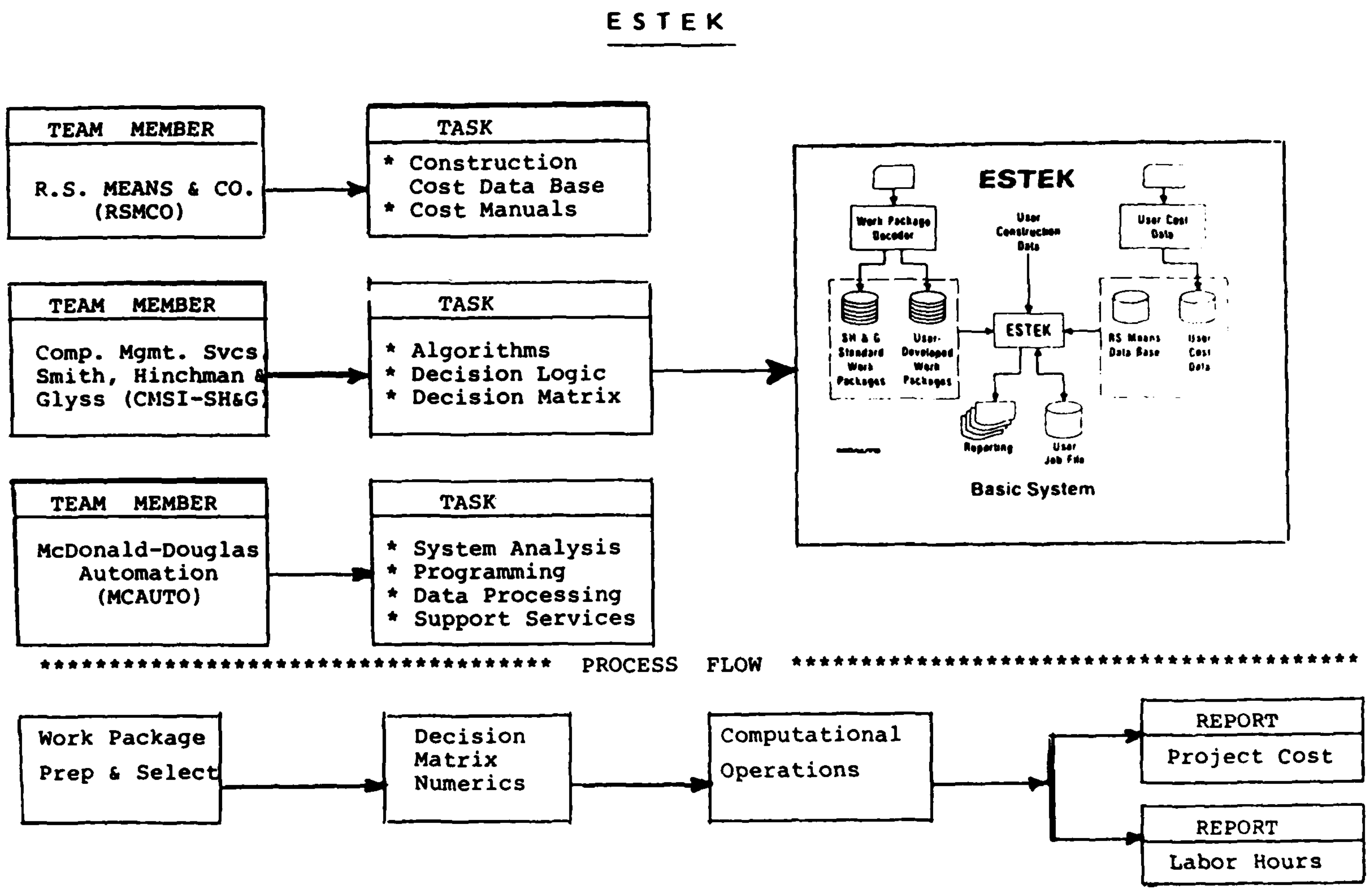


FIG.7.

CONSTRUCTION COST ESTIMATING MODEL (LEASABLE) FAST-C

DEVELOPER

F. FREIMAN - FREIMAN PARAMETRIC SYSTEMS (FPS)

(INVENTED/DEVELOPED RCA PRICE FAMILY OF MODELS)

DEVELOPMENT OF FAST-C FUNDED BY DoE, THE MAIN USER

A TRUE PARAMETRIC MODEL

HAS FOUR SEGMENTS

FAST-E

COMPUTES COST, DEVELOPMENT & PRODUCTION COST,

LARGE SCALE PLANTS & EQUIPMENT. USES COMMON DESIGN VARIABLES.

" THE ENGINEER'S MODEL "

FAST-C

COMPUTES CONSTRUCTION COST

OF PROJECTS, FACILITIES, BUILDINGS, DAMS, WATER COURSES, ETC.

FAST-M

COMPUTES COST OF TOTAL MINING OPERATIONS,

DEVELOPMENT, INSTALLATION AND OPERATION.

FAST-F

ALLOCATES RESOURCES VS SCHEDULES

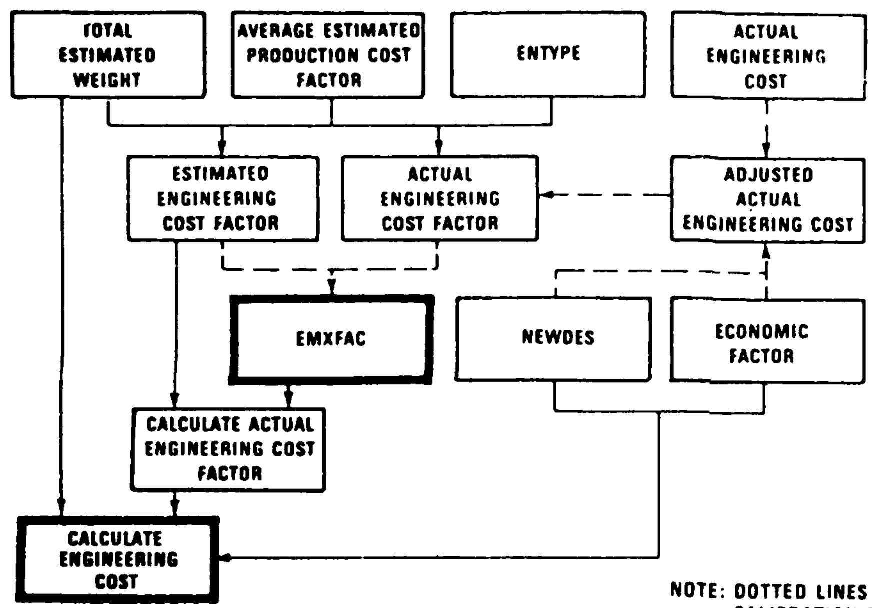
FOR ALL/ANY SEGMENT OF THE MODEL.

ALL SEGMENTS FULLY INTERACTIVE

FAST - FREIMAN ANALYSIS OF SYSTEMS TECHNIQUE.

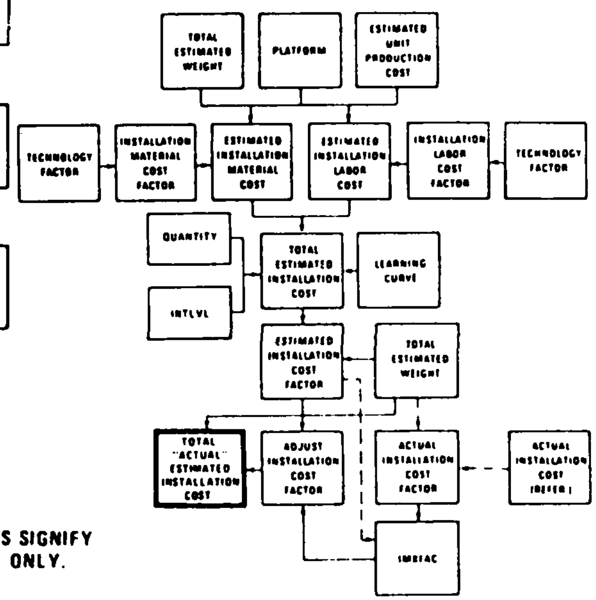
FIG.8.

HOW FAST-C WORKS



NOTE: DOTTED LINES SIGNIFY CALIBRATION ONLY.

FUNCTIONAL FLOW FOR ENGINEERING



FUNCTIONAL FLOW FOR INSTALLATION

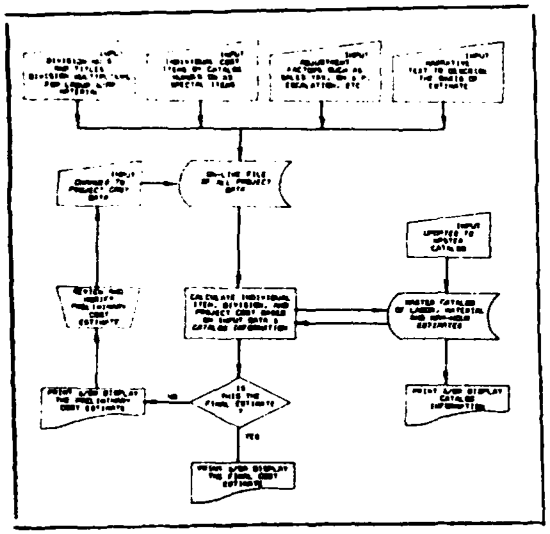
FIG.8A.

CONSTRUCTION COST ESTIMATING MODEL HSMM

DEVELOPER: HAYES, SEAY, MATTERN & MATTERN (HSMM)

(ARCHITECTURAL & ENGINEERING)

AVAILABLE UNDER CONTRACT



ACCURACY OF ESTIMATES VERSUS LOW BID

PROJECT	bid date	estimate $ millions	low bid $ millions
Electrical Distrib. System (USN Shipyard)	1/26/82	1.569	1.581
Parking Garage	9/28/81	3.383	3.212
Munitions Support Area	6/81	11.536	11.337
Munitions Ass'y Plant (U.S. Army)	3/80	27.653	27.774
Waste Incineration System (Municipal) Steam Generation	8/17/77	1.641	1.725
Airport Runways (Civil)	9/23/79	9.736	8.830
Creek Retention Basin (Municipal)	8/09/79	25.200	27.176

FIG. 9.

AD P001909

COST ESTIMATING FOR REGIONAL WATER SUPPLY STUDY

Marshall D. Lee
The Benham Group, Inc.
Oklahoma City, Oklahoma

The Water Supply Plan was part of the Tulsa Urban Study, which was a comprehensive analysis of problems, needs, and opportunities related to water resources in the Tulsa, Oklahoma, metropolitan area. The Tulsa Urban Study area includes, generally, the urban areas and potential urban areas surrounding Tulsa, Oklahoma. This area covers about 1,300 square miles in northeast Oklahoma. Included within the study area are 15 municipalities (of which 12 operate water utility systems) and 19 rural water districts providing water to approximately 520,000 people. The Tulsa Urban Study was conducted by the Tulsa District of the U.S. Army Corps of Engineers. Its purpose was to examine the broad and interrelated issues relating to water resources and to develop multipurpose plans to solve problems such as flooding and inadequate water supplies. The Tulsa District entered into a contract with The Benham Group, Inc., of Oklahoma City, Oklahoma, for the performance of the water supply portion of the Tulsa Urban Study. The objectives of the water supply portion of the study were to develop practical and cost-efficient plans for water supply sources, conveyance, and treatment facilities to meet the long-range needs of the study area.

In developing the water supply plans, the first step was to list all known alternatives. The alternatives consisted of the following:

Base Condition: Local governments and water suppliers would take no action either to limit demands for water or increase available supplies.

Water Conservation: The long-range effects of water conservation on reducing total demands were researched.

Groundwater Reuse: Water suppliers would use natural underground water supplies as sources.

Wastewater Reuse: The possibilities and effects of using wastewater to supply part of the thermal-electric power cooling water demand were determined.

New Storage Lakes: Possible new storage lakes were evaluated.

Change of Storage in Existing Lakes: Additional supplies could be made available by changing water quality, hydropower, or flood control storage to water supply storage in lakes in and near the study area. Such storage alterations would require a Corps of Engineers restudy of the lakes and subsequent action by Congress and, in some cases, might require physical modification of the dams or lakes.

Water Transfers: Use of the water from lakes where water supply storage is already available, would require that the water be transferred across basin boundaries. Construction of long-distance conveyance systems would be necessary.

Precipitation Augmentation: Local, state, or federal governments would use artificial techniques to change precipitation patterns and increase flow in area streams.

The base condition was held over for comparison purposes only. That plan would not meet the future needs of the area and is not an acceptable alternative. Groundwater use and precipitation augmentation were eliminated from further consideration as water supply alternatives. Groundwater in the study area lacks sufficient yields and is not of adequate quality to meet future municipal and industrial demands. Precipitation augmentation is still in the experimental stages. Its effects in areas dominated by convective and cyclonic cloud systems, such as the study area, are uncertain.

Water conservation, wastewater reuse, new storage lakes, changes in existing storage, and water transfers were all considered further. Individually, water conservation, wastewater reuse, and changes in existing storage would not meet all of the area's long-range needs. However, those measures in combination with other alternatives would make a difference in both future demands and availability of future sources, so they were carried over for further evaluation. New storage lakes and water transfers are viable alternatives and were held over for further study.

The existing alternatives allowed a range of options in sources for meeting the future water needs of the study area. Several plans were developed for management systems that could be used to develop sources, convey water from the source to the study area, and treat that water for distribution to users in the study area. Those management systems are as follows:

Base Condition: The 34 water-supplying entities in the study area would continue to supply water to their customers, using the current sources and treatment systems.

Independent Systems: Individual water supply systems would be developed for each community and rural water district, based on requested water rights and stated desires.

Subregional Systems: Groups of contiguous communities and rural water districts would develop joint conveyance and treatment facilities.

Regional Systems: One overall water supply system would be formed for each plan, using several sources and treatment facilities to serve the entire study area.

The initial analysis of water supply alternatives produced the following conclusions.

Under the base condition, the long-range needs of the study area would not be met. That condition should be retained in the study for comparison purposes, however, and redefined to include future actions that the various water suppliers are likely to take in the absence of any coordinated plan.

The independent system plan consists of 34 plans based on requested water rights or discussions with individual communities.

Both the subregional and regional systems would meet the future needs of the study area and should be retained for further consideration.

The planning process was an iterative one of formulating alternative plans, evaluating them, revising the alternatives, and reevaluating until the best plan could be identified. For the first iteration the initial step in formulation of development plans consisted of describing a variety of alternative plans for providing water service to all water service areas of the Tulsa urban area. Potential reservoir sites requiring extreme transmission distance or providing low yield were eliminated during the initial development of alternatives. Twenty-two subregional plans and four regional plans were evaluated, and three different projections of population and water demand were applied to each plan. Five subregions were assumed, with 1 to 10 water treatment plants in each subregion. Each plan included the facilities needed to deliver treated water to the communities served by the system. Distribution systems within the communities were not included. The facilities were sized to satisfy all three of the projections of demand to the year 2030.

The cost evaluation was accomplished by means of a computer program entitled Methodology for Areawide Planning Studies (MAPS). The MAPS program was developed by the Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi. It consists of a set of computer-based models that develop planning-level design and cost estimates for the facilities that make up a water supply system, including water treatment plants, storage tanks, pipelines, and pumping stations. The MAPS program was found to be reasonably accurate and produced results close to the actual engineering estimates of recently designed water supply systems.

The data entered into the MAPS computer program for the first iteration included general economic assumptions as well as the physical data pertaining to the treatment plants, pipelines, and other facilities.

The resulting estimates included capital costs and costs of operation and maintenance. The capital costs consisted of land and rights-of-way, construction, engineering and design, administration and inspection, and interest during construction. The operation and maintenance costs included electric power, chemicals for water treatment, labor, and materials. The cost of storage in federal reservoirs was added as a separate annual cost.

The pipeline costs were calculated for an array of pipe diameters in order to minimize the sum of the capital cost and the operating cost. Given the required flow rate and the pressure requirement, the program calculated the optimum pipe diameter and number of pumping stations for a water transmission facility.

The operating and maintenance costs of each facility were calculated on the basis of the average flow through that facility over the study period. All costs were converted to annual costs, then divided by the average annual water demand to calculate the cost per thousand gallons of water used. In

all, 78 different plans were entered into the MAPS computer program and evaluated as to the relative costs per thousand gallons of water consumed.

The first iteration of cost estimates indicated, generally, that subregional plans including only one water treatment plant could produce water at a lower cost than plans involving multiple treatment plants.

The results of the first iteration evaluation were presented to the Tulsa Urban Study Association committees and guidelines were adopted for the second iteration.

In the second iteration, the schematic plans were refined and the subregional boundaries were realigned in an effort to find the optimum combinations. Eight subregional plans and two regional plans were evaluated.

The comparative cost per thousand gallons of water was estimated for each second iteration plan, using the MAPS computer program as in the first iteration. It was found that the estimated overall costs for composites of subregional plans were essentially the same as the estimated costs of the regional plans.

In further analysis of the costs, the estimated cost of each plan was allocated to the participating communities by distributing the cost of each separate facility proportionally among the communities served by that facility. This method of allocation shows that 22 communities could obtain water at lower cost through a regional system, while 11 could be served at lower cost by subregional systems.

The subregions and regions evaluated in the third iteration were further refined after the second iteration and subregion boundaries were realigned. Three alternative plans were analyzed, each of which would serve the entire urban area. The estimated cost indicated that a combination of subregional plans were preferable to a regional plan for most of the communities. Three principal considerations were used in the evaluation: the cost effectiveness of the plans, the likelihood of implementation, and the ability of the plans to solve the problems identified for the area.

It was shown by the computer-generated cost estimates and the evaluation of alternatives that the best overall plan for water supply in the Tulsa Urban Area would involve six subregions. The location and capacities of facilities were completely reviewed. Pipeline routes were realigned to take advantage of public rights-of-way where practicable and to avoid terrain obstacles. Some of the facilities in the selected plan were omitted or deferred in the recommended plan where existing facilities could supply the demand for the year 2000.

The estimates of initial cost for the recommended plans were completely reviewed also, using the Methodology for Area Planning Studies (MAPS) computer-generated estimates in combination with recent experience in Oklahoma. The estimates included construction, contractor's overhead and profit, engineering, administration, interest during construction, and land, all at 1980 price levels. The estimates for water treatment plants included

clear-well storage and distribution pumping as well as the necessary treatment process equipment and buildings. The estimates for pipelines included pumping facilities and intake structures, where applicable, for delivery of water from the sources through the treatment plants to the general distribution areas. No water distribution mains, distribution storage tanks, or distribution booster pumps were included.

The final estimates were realistic and accurate, and the use of the program permitted investigation of a large number of alternatives. The final report included plans that would best serve the water supply needs of the Tulsa Urban Study area, using existing and potential water supply sources within a reasonable distance of the area.

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